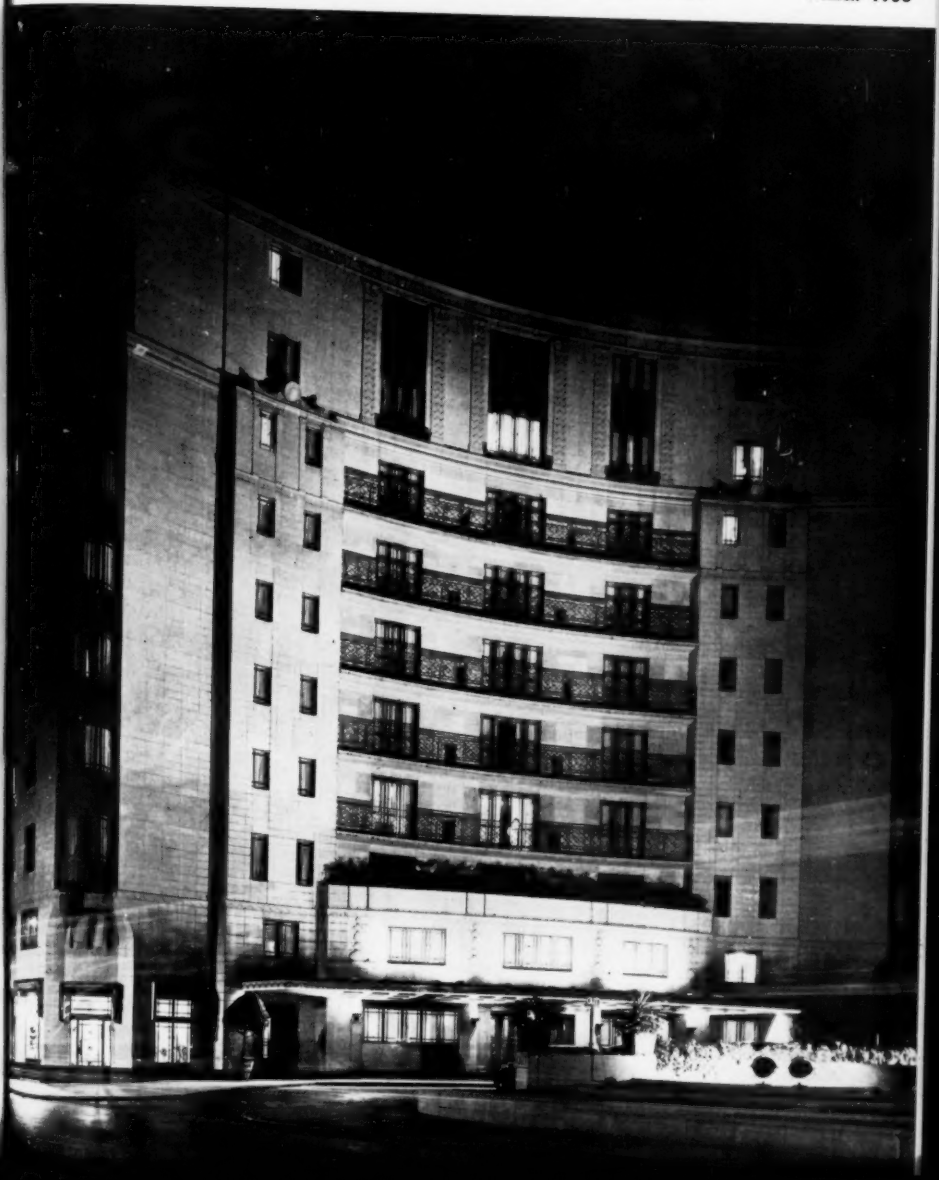
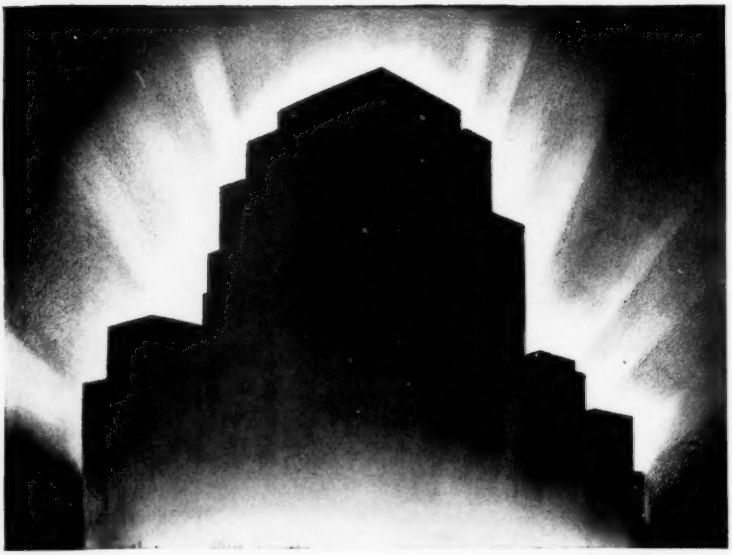
ACI News Letter

Vol. 21 No. 7 JOURNAL of the AMERICAN CONCRETE INSTITUTE March 1950





APPEARANCE...PLUS WHAT?

The first vital factor necessary to the construction of such massive buildings and other structures is a firm foundation.

Prepakt concrete "cast-in-place" piling, which is grouted from the bottom up, provides continuous support from bedrock to the structural footings. Where friction piles are indicated, the Prepakt grouting mortar penetrates the surrounding soil structure, providing literally thousands of contact bearing areas for each pile.

In many instances, Intrusion grouting alone has made actual concrete of the entire foundation material.

Prepakt representatives will gladly describe the advantages of Prepakt concrete and the engineering and field construction services available. Write or call the office nearest you.

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ACI

News Letter Contents

March 1950

On the Cover—Dorchester Hotel, Park Lane, London, by night. Even under this lighting the effective arrangement of the precast concrete facing slabs can be seen. For further information on British use of this technique see "Precast Concrete in Britain," p. 541, March JOURNAL.

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1950 Chicago checked off, Now it's "San Francisco, 1951"

The membership goal moves up—February 28, 1950—5013

These things were among those making the Chicago convention notable:

Registration—609 under curtailed travel conditions.

A general luncheon for 326 at which Frank Jackson was handed the gavel as the symbol of ascendancy to ACI presidency; tokens of award were presented to the year's awardees including "a first" Alfred E. Lindau award won by Frank E. Richart; an occasion throughout lighted by the characteristic zest and humor of retiring President Gilkey.

ACI's successful return to concurrent sessions, outjinxing the jinx of choice of which of two meetings to attend in one period; both pairs of the programs' simultaneous sessions had good and well-sustained audiences. Audiences were good from the first through the seventh and final program session, when a quiz session, so full of Chairman Scholer's fun and showmanship and the pungent distillate of concrete wisdom from his panel of eleven experts that pressure had to be brought to bear at five o'clock to reach a termination in the face of eager audience participation; many questions still remained in their capsules.

A public address system that worked.

Meetings of 18 technical committees or subcommittees Monday morning, evening and Tuesday evening—no other planned events after the dinner hour.

The convention weather? Variably dull, wet and soggy to brilliant, dry and windy.

An unusually solid program packed many diverse subjects; concurrent sessions through two full periods increased program time and scope by 40 percent.

Who's Who This Month

Frank H. Jackson

Frank H. Jackson, ACI President, is author of "A Way to Better Pavement Concrete," p. 489. Mr. Jackson, an ACI member since 1924, needs no introduction here to JOURNAL readers. Author or co-author of seven ACI papers in the past 20 years, he was awarded with Harold Allen the Wason Medal for the most meritorious paper in 1948 for "Concrete Pavements on the German Autobahnen," ACI JOURNAL, June, 1948.

Mr. Jackson has served as ACI vice-president as well as on numerous technical committees in addition to publications, advisory, program and technical activities committees. He has been associated with the Bureau of Public roads since 1905 and is now its Principal Engineer of Tests. For his outstanding work in the highway field he was awarded the Distinguished Service Award of the Highway Research Board in 1948 and in 1950 received the Meritorious Service Award of the U. S. Dept. of Commerce.

W. S. Colby

Drawing on his experience with Stone and Webster Engineering Corp. where he is a structural engineer, W. S. Colby describes the "Design and Construction of a Circulating Water Intake," p. 497. Starting in 1924 as a tracer with Stone and Webster he subsequently became draftsman, checker, engineer in the structural division and structural engineer. He has also worked for short intervals as structural engineer for McClintock and Craig, Springfield, Mass., United Engineers and Constructors, Procurement Division of U. S. Treasury Dept., U. S. Army Engineers on the Passamaquoddy Project, Eastman Kodak Co., and TVA.

A 1925 graduate of Massachusetts Institute of Technology with a B. S. in Civil Engineering, Mr. Colby is a member of the Boston Society of Civil Engineers, A.S.C.E. and the Massachusetts and National Professional Engineers Societies.

Donald C. Andrews and Nomer Gray

Donald C. Andrews, co-author of "The Contractor's Viewpoint of Inspection," p. 509, has been since 1941 in general supervision of construction work for Turner Construction Co. He held various field positions with this firm between 1925 and 1941.

A civil engineering graduate of McGill University, Montreal, Canada, he is a Professional Engineer, State of New York, and is a member of ACI and the "Moles."

Nomer Gray is presently development engineer with Walter Kidde Constructors, Inc., one of the oldest building construction firms in the East.

After spending some 15 years in the field of heavy construction projects including highways, sewers and a variety of bridges, in positions ranging from inspector to division engineer, he entered the office to work on the administrative phase of large engineering projects handling problems in procedure, cost accounting and personnel work.

A special assignment of unusual interest in recent years was a two year comprehensive study of the famous old Brooklyn Bridge in New York which led to the reconstruction now under way.

Mr. Gray holds two degrees in civil engineering from Polytechnic Institute of Brooklyn and has taught there. He has been active in ACI, A.S.C.E., and Municipal Engineers of the City of New York. He now serves on ACI Committee 611, Inspection, and is a member of Tau Beta Pi and Sigma Xi.

Pletta, Massie and Robins

D. H. Pletta, co-author of "Corrosion Protection of Thin Precast Concrete

Sections," p. 513, is Head of the Applied Mechanics Dept. and Director of the Materials Testing Laboratory at Virginia Polytechnic Institute, and has been active in ACI activities since 1931 when "Permeability of Gravel Concrete" was published with P. T. Norton as co-author. His "Notes on Existing Rigid Frame Bridges" (1938) developed out of his membership on Committee 314, Rigid Frame Bridges, and in 1939, "Beams With Intermediate Expansion Hinges in Rigid Frame Bridges" was published with L. C. Hollister as co-author. His chairmanship of Committee 314 from 1940-42 and 1947 to date, was interrupted by Army service as a Major in the Ordnance Dept. as Officer-in-charge, Richmond Regional Office, Philadelphia Ordnance District, in charge of the procurement, inspection and production of over $20,000,000 in ordnance material, and on the faculty at the Military Academy at West Point.

He received his B.S. in C.E. (1927) and C.E. Degree (1938) from the University of Illinois, and an M.S. in C.E. (1931) from Wisconsin. Since graduation, he has spent a number of years in design and construction, and the remainder in research and teaching.

E. F. Massie is a stress analyst with Montague and Betts, Steel Fabricators, Lynchburg, Virginia. He received his B.S. in C.E. in 1948 and his M.S. in Applied Mechanics in 1949, both from V.P.I. after spending 3 years in the Army. During his graduate year, he was a research fellow in Applied Mechanics assigned to this Navy Project.

H. S. Robins is a research fellow at Virginia Polytechnic Institute working on his M.S. in Applied Mechanics and is currently employed in the Navy Research Project reported in this paper. He received his B.S. in C. E. from V.P.I. in 1949, has had 2 years of experience in concrete construction, and spent 4 years in the Army, being discharged as a Captain in the Engineer Corps.

Paul Rogers

As background for his "Design of Hexagonal Bins," p. 529, Paul Rogers has 16 years of engineering experience in this country and in Transsylvania where he was born. He is presently design engineer with Sargent and Lundy, Engineers, Chicago, and has also worked as structural engineer with Ralph H. Burke, Airport Consultant, Roberts and Schaefer Co., Hiram Walker and Sons, Inc., A. J. Boynton and Co., Publicity Utility Engineering and Service Corp. and Three Sisters, Inc. He has also served as consulting engineer on a number of projects in this country and in Transylvania.

A graduate civil engineer from the Ecole du Gènie Civil, Paris, Mr. Rogers also studied in the Graduate School, Illinois Institute of Technology. He was assistant professor of structural engineering at the Chicago Technical College from 1940-45 and is the author of "Two-Way Reinforced Concrete Slabs," ACI JOURNAL, Sept. 1944.

Mr. Rogers is a registered Structural and Professional Engineer in Illinois and is a member of ACI, A.S.C.E., Western Society of Engineers, American Society for Engineering Education and the American Civic and Planning Assn.

P. G. Bowie and A. R. Collins

The authors of "Precast Concrete in Britain," p. 541, are both associated with the Cement and Concrete Assn., London, England.

P. G. Bowie, after qualifying as an engineer in 1907, joined the Waring White Building Co., contractors for the Selfridge Store and the Royal Automobile Club buildings. He was later designer with the Indented Bar Co. and with various contractors engaged in the erection of both steel frame and reinforced concrete buildings. In 1910 he was co-author with Oscar Faber of a book on reinforced concrete design. For the last 15 years he has been chief structural engineer of the Cement and Concrete Assn.

Continued on p 15

Use This Improved Bar FOR FIRM ANCHORAGE IN CONCRETE

In ordering reinforcing bars be sure to call for bars with deformations designed to provide maximum bond strength in concrete—reinforcing bars as outlined in Specification A 305-49.

This specification, which was recently adopted by the American Society for Testing Materials, calls for (1) deformations of specified height, to be spaced at uniform distances, (2) deformations to be placed with respect to axis of bar so that the included angle is not less than 45 deg, (3) average spacing between the deformations not exceeding seven-tenths of the bar diameter, and (4) deformations to be of such length that the gap existing between the ends of the deformations is not more than 12½ pct of the perimeter of the bar.

The improved Bethlehem Reinforcing Bar provides exceptionally high bond strength because it more than meets these requirements of Specification A 305-49. The bar has high, closely spaced ribs which provide firm anchorage between steel and concrete. This means low slips at working loads, which helps prevent the formation of wide tensile cracks in the concrete. Result: More efficient, more economical, neater-looking structures.

The Bethlehem Reinforcing Bar is made from new-billet steel in all sizes from ⅜ in. to 1¼ in. Specify it for your next concreting job.

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Bethlehem Reinforcing Bars

Honor Roll

February 1, 1949, to January 31, 1950

The close of the current Honor Roll finds the Morgans, father and son, leading the list with combined credit for 83½ new member applicants. Professor F. N. Menefee, University of Michigan, and Jose Antonio Vila, Havana, Cuba, are tied for third place honors with 11½ credits each.

February 1 marks the beginning of our new Honor Roll and we are hopeful that even a larger percentage of our Members will participate actively in introducing new Members to ACI in the coming year.

Newlin D. Morgan (Ill.) 55½
Newlin D. Morgan, Jr. (Wyo.) 28
F. N. Menefee (Mich.) 11½
Jose Antonio Vila (Cuba) 11½
R. H. Sherlock (Mich.) 9
H. J. Gilkey (Ia.) 7½
Raymond E. Davis (Calif.) 7
James A. McCarthy (Ind.) 7
F. J. Ochoa U. (Guatemala) 7
Henry E. Griset (N. C.) 6½
Howard Simpson (Mass.) 6½
Jaime Alberto Mitrani (Cuba) 6
Roberto Barrillas F. (Guatemala) 4½
William T. Neelands (Ga.) 4½
J. W. Kelly (Calif.) 4
Luis A. Pietri Lavie (Venezuela) 4
William D. Nowlin (Va.) 4
J. Vicente Orozco (Mexico) 4
H. F. Gonnerman (Ill.) 3½
John J. Hogan (N. Y.) 3½
R. R. Kaufman (Ohio) 3½
Ralph W. Kluge (Ind.) 3½
Frank E. Richart (Ill.) 3½
Chester P. Siess (Ill.) 3½
Thomas E. Stanton (Calif.) 3½
Stanton Walker (D. C.) 3½
J. F. Baxter (N. Y.) 3
Elmo C. Higginson (Colo.) 3
J. E. Jellick (Calif.) 3
H. D. Jolley (Ill.) 3
Barton Jones (Puerto Rico) 3
Frederic F. Mavis (Pa.) 3
A. J. McElrath (Tenn.) 3
D. H. Pletta (Va.) 3
T. W. Thomas (Minn.) 3
Leon Venegas (La.) 3
Sterling Lowe Bugg (Fla.) 2½
Harold S. Carter (Utah) 2½
R. A. Caughey (Ia.) 2½
Roger H. Corbetta (N. Y.) 2½
Carl A. Menzel (Ill.) 2½
Edward W. Scripture, Jr. (Ohio) 2½
George Winter (N. Y.) 2½
Paul W. Abeles (England) 2
A. Amirikian (Md.) 2
H. J. Bezette (Wash.) 2
Adraino Carbone (India) 2
R. E. Copeland (Ill.) 2
W. S. Cottingham (Wis.) 2
Jacob J. Creskoff (Pa.) 2
John G. Dempsey (N. Y.) 2
Ambrosio R. Flores (Calif.) 2
Zorislav Franjetic (Argentina) 2
Walter N. Handy (Ill.) 2
Robert J. Hansen (Mass.) 2
Myle J. Holley, Jr. (Mass.) 2
V. P. Jensen (Calif.) 2
Emory E. Johnson (S. D.) 2
Edgar R. Kendall (Nebr.) 2
Henry L. Kennedy (Mass.) 2
K. V. Kini (India) 2
Karl W. Lemcke (N. Y.) 2
Sam McCluer (Ga.) 2
H. J. McGillivray (Fla.) 2
Rafael Garcia Montes (Cuba) 2
William S. Moore (Ind.) 2
George G. Nichols (Ore.) 2
LeRoy T. Oehler (Texas) 2
Luis Paredes-Manrique (Colombia) 2
Charles Peterson (Calif.) 2
Wilfrid Schnarr (Canada) 2
Thomas C. Shedd (Ill.) 2
Frederick E. Springate (Calif.) 2
Manabu Tanaka (Hawaii) 2
G. E. Troxell (Calif.) 2
John H. Bateman (La.) 1½
R. Y. Bush (Va.) 1½
S. J. Chamberlin (Ia.) 1½
Leo H. Corning (Ill.) 1½
H. C. Delzell (Ill.) 1½
Peter J. Doanides (S. Africa) 1½
A. H. Douglas (Canada) 1½
Adly W. Gindy (Egypt) 1½
Adli Milik Hanna (Egypt) 1½
Frank Kerekes (Ia.) 1½
R. G. King (Ia.) 1½
Robert L. Mauchel (N. Y.) 1½
Harmon S. Meissner (Colo.) 1½
Roy T. Sessums (La.) 1½
J. Morgan Smith (Ga.) 1½
Bertrand H. Wait (N. Y.) 1½
Calvin T. Watts (La.) 1½
William R. Waugh (D. C.) 1½
E. C. Wenger (Ill.) 1½
Frank W. Wheeler (Va.) 1½
A. J. Abrams 1
Jerome O. Ackerman 1
William F. Anderson 1
Harold Austin 1
Samuel Baker 1

J. A. Bakker 1
Charles F. Ball 1
Hugh Barnes 1
Donald C. Beam 1
Herman A. Bell 1
Oscar Benedetti 1
Robert F. Blanks 1
Hans H. Bleich 1
Ernest M. Bowles 1
R. W. Brandley 1
Carlos Mora Bravo 1
C. W. Britzius 1
Frank B. Brown 1
Milton Brumer 1
D. R. Cervin 1
R. V. Chate 1
H. Clare, Jr. 1
Miles N. Clair 1
E. F. J. Clark 1
H. W. Comack 1
Alirio Cornejo 1
Hendrik Cornelissen 1
R. W. Coyle 1
G. H. C. Crampton 1
Ray A. Crysler 1
Jose A. Cuevas 1
M. H. Cutler 1
E. Davis 1
Harmer E. Davis 1
W. Dewoody Dickinson, Jr. 1
A. T. Docal 1
Ernest A. Dockstader 1
W. H. Droege 1
C. Martin Duke 1
Clarence W. Dunham 1
Belmon U. Duvall 1
John J. Dwyer 1
Harlan H. Edwards 1
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F. L. Fitzpatrick 1
George W. Ford 1
Carl Fox 1
Robert W. Freeman 1
Carmelo Galindez 1
Gerald K. Gillan 1
Castor Segunda Goa 1
Ali N. Gokyigit 1
E. Gonzalez-Rubio 1
Armour T. Granger 1
Samuel J. Green 1
J. E. Guest 1
K. Hajnal-Konyi 1
Robert C. Hanckel 1
George E. Hatch 1
Bernard C. Herring 1
M. D. Hicklin 1
Percy Howat 1
Franklin D. Howe 1
Myron A. Howe 1
J. W. Hubler 1
Frank H. Jackson 1
Alberto Dovali Jaime 1
J. P. Jain 1
M. E. James 1
Bruce Jameyson 1
V. P. Jensen 1
R. C. Johnson 1
Oliver G. Julian 1
Milton Kellert 1
Reginald A. Kirkpatrick 1
Alexander Klein 1
Max Krueger 1
Viljo Kuuskoski 1
Blas A. Lamberti 1
George E. Large 1
William Lerch 1
Harry E. Lewis 1
Leslie D. Long 1
Winthrop E. Luke 1
E. Lyons 1
Charles Macklin 1
Ian MacTaggart 1
Leon C. Magbanua 1
Glenway Maxon 1
D. M. McCain 1
E. C. McClintock, Jr. 1
George C. McLeod 1
F. R. McMillan 1
M. J. McMillan 1
Jesus Mesa 1
J. E. Miller 1
Eugene Mirabelli 1
Robert B. B. Moorman 1
Robert Morris 1
R. F. Moss 1
Harold H. Munger 1
E. L. Oliver 1
George L. Otterson 1
Czeslaw Ozieblowski 1
C. C. Parker 1
Orin G. Patch 1
Jose F. Perez 1
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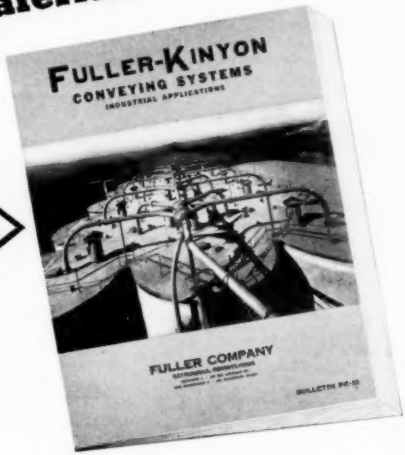
Anthony F. Pessolano 1
A. Edward Piller 1
J. D. Piper 1
James M. Polatty 1
W. S. Powell 1
Emil Praeger 1
Gayle B. Price 1
Evan L. Richard 1
Raul Miguel Rivero 1
J. E. Robinson 1
Paul Rogers 1
Georges Ronai 1
Albert Rootberg 1
William A. Rose 1
John A. Ruhling 1
Carl E. Sandstedt 1
Emil Schmid 1
Philo H. Schultz 1
F. Selmer 1
Saul Shaw 1
Isaac S. Shina 1
Ranbir Singh Shrikant 1
Richard Skalak 1
Myron M. Smith 1
William Furber Smith 1

Continued on p. 10

This bulletin is full of ideas how to handle Portland cement and other dry pulverized materials

BULLETIN FK-25



Bulletin FK-25, a 24 page booklet, illustrates and describes the Fuller-Kinyon Conveying System as applied to industry for the unloading and conveying of Portland cement and other materials. It shows, by line drawings and photographs, its application to material handling in asbestos-shingle manufacturing plants, ready-mix concrete plants, phosphate plants, barge and ship unloading, fly-ash handling in power plants, as well as its application in Portland cement manufacturing plants.

Just ask for Bulletin FK-25, and it will be sent you promptly.

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P-115

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PASSAIC, N. J.

Sika

Honor Roll

Continued from p. 8

Luis G. Solano A. 1
Leroy A. Staples 1
Alois Starkmann 1
Martin Steljes 1
Jiri Stork 1
W. B. Summer 1
E. O. Sweetser 1
Maurice Taylor 1
A. G. Timms 1
I. L. Tyler 1
Oscar J. Vago 1
Joseph J. Waddell 1
Ralph A. Wakeman 1
David Watstein 1
John M. Wells 1
J. G. Welsh 1
John Williamson 1
Donald K. Woodin 1

The following credits are, in each instance "50-50" with another member.

Robert F. Adams
A. L. Alin
Birger Arneberg
Michel Bakhoum
C. Harold Barcus
Charles H. Barker
Frank Baron
Frank J. Barrett
J. F. Barton
J. E. Bates
Aaron H. Bauman
E. W. Bauman
Fred Beatty
H. W. Beckingham
A. H. Brewer
Harold W. Brewer
A. R. Brickler
A. A. Brielmaier
Ralph A. Britson
Robert G. Burnett
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Napoleon V. Campomanes
W. R. Chandler
Anthony Henry Clark
Arthur P. Clark
Leo Coff
Sidney P. Cohen
T. F. Collier
R. D. Conrow
W. A. Coolidge
Dwight A. Covington
William A. Crabb
William P. Crawford
Lauro M. Cruz
A. E. Cummings
W. A. Davis
Paul C. Disario
Curzon Dobell
Jay C. Ehle
W. J. Emmons
J. L. Faisant
A. V. Farley
Phil M. Ferguson
Jorge Figuls
Benjamin Forsyth
James J. Fox
Daniel Frederick
J. N. Galli
Ian Glauber
Norman Alexander Grandage
Walter Griesbach
A. W. Haddow
Homer M. Hadley
Stanley P. Haendel
J. R. G. Hanlon
W. S. Hanna
Milton Hartstein
M. J. Hawkins
John M. Hayes
James A. Hedges
R. G. Hennes
James A. Highsmith, Jr.
Frank R. Hinds
A. E. Hiscox
Samuel Hobbs
Eivind Hognestad
S. C. Hollister
Albert C. Hooke
S. E. Hyde, Jr.
Melvin Fu Hoon Ja
Paul E. Jeffers
Bruce M. Johnson
Samuel Judd
B. D. Keatts
John C. King
Lane Knight
Carl A Koerner
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A. E. Mackney
M. F. Macnaughton
Robert M. Mains
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Henri Marcus
Hudson Matlock
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H. E. A. McCarthy
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Sylvester Morabito
William Morris
William H. Munse
S. Neatwait
Philip M. Noble
Paul W. Norton
Calvin C. Oleson
Chester B. Palmer
Douglas E. Parsons
Joe Peeler Patrick
George Nelson Perry
Carlyle W. Peterson
E. O. Phlegar
Alan H. Pilling
Raymond G. Post
Herman G. Protze
Warren Raeder
Irvin S. Rasmusson
F. V. Reagel
Allan M. Reese
Raymond C. Reese
Francisco M. Rexach
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H. H. Roberts
Paul Rogers
William H. Rowan
Fred H. Ryan
Cecil F. Schaaf
Leo W. Schmidt
John F. Shea
G. G. Shimamoto
Jack S. Short
Ruben Ruiz Silva
Tirlochan Singh
Einar Skinnarland
Charles A. Snyder
H. F Sommerschield
R. A. Spencer
Frank L. Stahl
Peter M. Stewart
G. W. Stokes
E. Q. Sullivan
S. Szczerbaniewicz
Anton Tedesko
William H. Thoman
J. Trueman Thompson
E. W. Thorson
Victor Thorson
LeRoy A. Thorssen
Homer Thrall
Robert E. Tobin
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Jose E. Toledo S.
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John Tucker, Jr.
Maxwell Upson
Rudolph C. Valore, Jr.
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Norman Wagner
W. Wayne Wallace
M. H. Walters
David J. Warsaw
George W. Washa
J. C. Watt
Frederic N. Weaver
Carl Weber
J. Wegrowski
Jack D. White
Isabel Whitmarsh
Charles S. Whitney
Henry Wilckens
Benjamin Wilk
C. D. Williams
H. F. Williams
Paul C. Winters
M. O. Withey
William H. Wolf
Hubert Woods
Thomas Worcester
K. W Wright
Charles E. Wuerpel
Alexander H. Yeates
R. B. Young
George D. Youngclaus
Robert Zaborowski

Positions and Projects — ACI Members

Another milestone

With 5013 members as of March 1, ACI passes on to new membership goals. Continued effort by all members should make easier the task of acquainting others in the concrete field with the benefits of Institute affiliation.

Prestressed concrete discussed

At a joint meeting of A.S.C.E. and the Western Society of Engineers in Chicago, February 6, Blair Birdsall, assistant chief engineer of John A. Roebling's Sons Co. bridge division, gave a talk on application of American methods and processes. A movie on this subject prepared by Preload Enterprises, Inc. was shown.

Engineers meet in California

Highway engineers met recently at two conferences sponsored by the Institute of Transportation and Traffic Engineering, University of California. On January 30, a three-day conference, popularly known as the "road school" began at Los Angeles where broad planning problems, administration, design, construction and maintenance were discussed. On February 3 at Berkeley a one-day conference considered the design of urban arterial highways.

Rice, Stokes and Ferguson

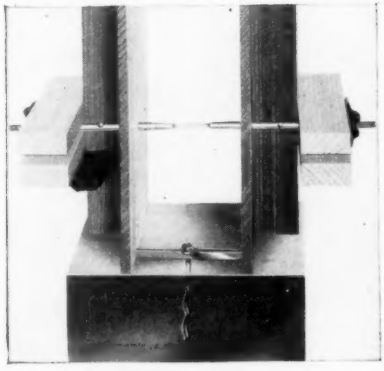
The office of John W. Rice, Consulting Engineer, Pittsburgh, has been expanded and reorganized under the new name of Rice, Stokes and Ferguson. Among other engineering services the firm offers structural design, preparation of plans and specifications and supervision of construction.

Zement-Kalk-Gips

For the German-language publication *Zement-Kalk-Gips* a volunteer reviewer is needed. This monthly magazine deals mainly with the technical details of production and research in connection with cement, lime and gypsum. Most of the articles contain many chemical terms. Interested persons please write to the ACI Secretary for further details.

Do you read Japanese?

If so, the Institute headquarters has a number of Japanese treatises published by the Japan Society of Civil Engineers which are in need of review. They will be sent to the first persons indicating an interest in reading them.



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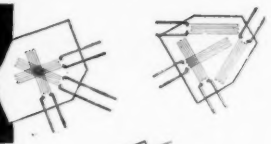

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SR-4 GAGES TEST PRESTRESSED CONCRETE GIRDER. This prestressed girder, a duplicate of those which U.S. Preload Corp. will furnish for a new Philadelphia bridge, was tested to destruction under 11 times designed load. Baldwin SR-4 gages gave an accurate progress report of stresses throughout the test.

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TESTING HEADQUARTERS **BALDWIN**

New Members

With 85 new member applicants approved for the month of January the total membership became 4933 on January 31, 1950.

Among the 85 listed there are 47 Individuals, 3 Corporations, 14 Juniors and 21 Students.

Geographically, Illinois leads the list of January newcomers with 10; California is in second place with 8.

Arizona

Burdett, Harold D., (Indiv.) Box 6276 Capitol Station, Phoenix, Ariz.
Ferguson, Ben H., (St.) 85 Holly St., Phoenix, Ariz.
Hoyt, C. Leroy, (Indiv.) P. O. Box 423, Ray, Ariz.

California

Abrams, James D., (Jr.) Bureau of Public Roads, c/o American Embassy, APO 736, c/o P. M., San Francisco, Calif.
Beiser, Alvin Phillip, (Jr.) One El Centro Ave., San Mateo, Calif.
Eagle, Harold L., (St.) 355 S. 17th St., Apt. 2D, Richmond, Calif.
Fairman, Gibson Willard, (St.) 308B S. Freeman Ave., Inglewood, Calif.
Forrest, Gordon R., (St.) 1829E Berkeley Way, Berkeley 3, Calif.
Nash, Robert A., (Indiv.) 22 S. Baldwin Ave., Sierra Madre, Calif.
Schwartz, Simeon Alan, (St.) 617 N. Cherokee, Los Angeles 4, Calif.
Sexton, H. J., (Indiv.) 933 Millie Ave., Menlo Park, Calif.

Colorado

Howley, Charles Bertrand, (Indiv.) 25 E. 16th Ave., Denver, Colo.

Connecticut

Anderson, Arthur R., (Indiv.) 110 Woodbury Ave., Springdale, Conn.
Muktabhant, Chai, (St.) 202 Park St., New Haven, Conn.

District of Columbia

Green, William Charles, (Jr.) National Bureau of Standards, Washington 25, D. C.
Minear, V. L., (Indiv.) Dept. of the Army, O. C. E., Civil Works, Washington 25, D. C.

Florida

Bowler, John L., (Indiv.) 2784 Downing Ave., Jacksonville 5, Fla.

Georgia

Fain, C. S., (Indiv.) McRae, Ga.
Patterson, J. W., (Indiv.) 263 Decatur St. S. E., Atlanta, Ga.
Waters, Robert Ollie, Jr., (Indiv.) c/o Cason J. Callaway, Hamilton, Ga.

Illinois

Albert, Edward R., (Indiv.) 120 S. LaSalle St., Chicago 3, Ill.
Cherry, Sheldon, (St.) 114a Talbot Laboratory, University of Illinois, Urbana, Ill.
Collins, Roland Dean, (St.) 57 E. Healey, Champaign, Ill.
Eckhoff, Harold Eugene, (Jr.) 2415 Park Ave., Cairo, Ill.
Miller, Thomas Cornelius, (Indiv.) 335 W. 64th St., Chicago 21, Ill.
Tholin, A. L., (Indiv.) 188 W. Randolph St., Room 2400, Chicago 1, Ill.
Walls, Cecil Albert, (Indiv.) Portland Cement Association, 33 W. Grand Ave., Chicago 10, Ill.
Wei, Benjamin Chih-Fang, (Jr.) 105 Talbot Laboratory, University of Illinois, Urbana, Ill.
Wiernasz, E. J., (Jr.) 8515 S. Yates, Chicago, Ill.
Wright, Douglas Tyndall, (St.) 114a Talbot Laboratory, University of Illinois, Urbana, Ill.

Iowa

Hurst, Milford L., (Indiv.) 827 Western Ave., Waterloo, Ia.
Ibrahim, Amin Aly, (Indiv.) Agricultural Engineering Dept., Iowa State College, Ames, Ia.
Williams, Roger F., (Indiv.) 3420 S. W. 9th St., Des Moines, Ia.

Massachusetts

Iwata, Samuel Isamu, (St.) 8 Blossom St., Boston 14, Mass.
Lukashok, Alvin, (St.) 400 Commonwealth Ave., Boston 15, Mass.
Main, Inc., Chas. T., (Corp.) 80 Federal St., Boston, Mass. (R. A. Moncrieff)

Michigan

Borin, Ralph, (Indiv.) 19837 Monte Vista, Detroit, Mich.
Myers, Melvin L., (Indiv.) 401 Grace St., Adrian, Mich.

Minnesota

Gardner, Elza Loyd, (Indiv.) 1362 Mackubin St., St. Paul, Minn.

Johnson, Rodney W., (St.) 3126 4th St. S. E., Minneapolis 14, Minn.

Missouri

Grant, Nathaniel, (Indiv.) 807 W. 48th 702, Kansas City 2, Mo.

New Jersey

Callahan, Frank T., (St.) 407 Second St., Swedesboro, N. J.
Gruzen, Benjamin Macson, (Indiv.) 96 Plymouth Ave., Maplewood, N. J.

New York

Babcock, Henry Nash, (Jr.) c/o Skinner, Cook & Babcock, Inc., 60 E. 42nd St., New York 17, N. Y.
Garfinkel, Aaron, (Indiv.) 1150 Brighton Beach Ave., Brooklyn 24, N. Y.
Frederic Harris, Inc., (Corp.) 27 William St., New York 5, N. Y. (Edward J. Quirin)
Pranich, Kanok, (Jr.) 144 S. Geneva St., Ithaca, N. Y.
Shields, Lavelle W., (Indiv.) 441 Lexington Ave., New York 17, N. Y.
Tonetti, Nella, (Indiv.) Cooper Union Library, Cooper Square, New York, N. Y.

North Carolina

Dellinger, Edgar Streetford, (St.) P. O. Box 5091, State College Station, Raleigh, N. C.
Sapp, Robert Alvyn, (St.) 10 Enterprise St., Raleigh, N. C.

Ohio

Fowler, A. B., (Indiv.) Midland Building, Cleveland 15, Ohio
Longenecker, James R., (Indiv.) 535 N. High St., Covington, Ohio
Powers, Robert W., (Indiv.) 1855 W. Market St., Akron 13, Ohio

Tennessee

Horner, Frank, (Jr.) Box 312, Waverly, Tenn.
Stites, John Dudley, (Indiv.) c/o Builders Supply Co., Cookeville, Tenn.

Texas

Christian, G. L., (Indiv.) P. O. Box 2531, Amarillo, Texas
Stanovsky, Joseph J., (St.) 204 Archway, Apt. 1, Austin, Texas
Weatherbee, Louis B., (Indiv.) 4236 Bryn Mawr Dr., Dallas 5, Texas

Virginia

Blake, Oscar J., (Jr.) 2421 Brandon Ave., S. W., Roanoke, Va.
Davis, George B., (Jr.) Box 72, Bassett, Va.
Kinnier, Henry L., (Indiv.) Thornton Hall, University of Virginia, Charlottesville, Va.
Page, John R., (Indiv.) P. O. Box 72, Bassett, Va.

Washington

Mason, William Ralph, (Indiv.) University of Washington, More Hall, Seattle 5, Wash.

Wyoming

Strid, Lloyd H., (St.) 100 Doolittle St., Laramie, Wyo.

Puerto Rico

Arbogast, C. B., Jr., (Jr.) 1419 Las Palmas, Stop 20, Santurce, Puerto Rico
Vincenty, Rafael, (Indiv.) Box 2749, San Juan, Puerto Rico

Canada

Brison, John W., (Indiv.) 385 Askin Blvd., Windsor, Ont., Canada
Fowler, Charles A. E., (Indiv.) c/o C. A. Fowler & Co., Capitol Bldg., Halifax, N. S., Canada
Howard, John W., (Indiv.) c/o Howard Sand & Gravel Co., Ltd., Aldershot, Ont., Canada
King, J. C., (Indiv.) Regional Bridge Engineer, Canadian National Railways, Moncton, N. B., Canada
Massell, Chas. B., (Jr.) 85 Gainsborough Rd., Toronto, Ont., Canada

Cuba

Chomat B., Jose Roberto, (Indiv.) Escuela de Arquitectura, Universidad de La Habana, La Habana, Cuba
Pujol, Juan B., (Indiv.) Hospital No. 632, Havana, Cuba

England

Lau, Yat-Sun, (St.) 12 Wetherby Place, Kensington, London S. W. 7, England
Pyrene Co., Ltd., (Corp.) Great West Road, Brentford, Middlesex, England
Thornton, John H., (Indiv.) 29 St. Albans Rd., Hatfield, Hertfordshire, England

France

Wach, F. E., Jr., (Indiv.) Dewalco S.A.R.L., 14 Ave. Hocke, Paris 8, France

Guatemala

Carrera, Rene Castillo, (St.) 28 Ave. & 6a. C. No. 25, "La Palmita," Guatemala City, Guatemala
Medina M., Manuel, (St.) 3a Calle Oriente No. 37A, Guatemala, Guatemala

Vides T., Amando, (St.) Facultad de Ingenieria, Universidad de San Carlos, 8a Avenida Sur No. 18, Guatemala, C. A.

India

Ghosh, Bijitendra Nath, (Jr.) 93/2 Bakul Bagan Rd., Bhowanidore, Calcutta, India

Shroff, Jehangir C., (Indiv.) Prospect Chambers Annexe, Hornby Road, Fort Bombay, India

Philippines

Carrillo, Abelardo B., (Indiv.) Bureau of Public Works, Manila, Philippines

S. Africa

Reid, W. R., (Indiv.) The Hume Pipe Co. (S. Africa) Ltd., P. O. Box 204, Germiston, Transvaal, S. Africa

Who's Who

Continued from p. 5

engaged in pointing the way to other engineers.

A. R. Collins graduated in civil engineering in 1933 from the University of London and after two years on engineering projects, worked first at the Building Research Station and later at the Road Research Laboratory of the British Department of Scientific and Industrial Research. Except for a few years during the war when he was engaged on wartime research including work preparatory to the attack on the Mochne and Eder dams he has spent the whole of his time with the D.S.I.R. on concrete research, principally in connection with mix design and frost resistance for which he obtained his D.Sc. degree in 1949.

In 1947 he joined the Cement and Concrete Assn. and is now Technical Director.

Multi-story precast concrete buildings

A method of constructing multi-story precast concrete buildings has been developed by Karl P. Billner, Vacuum Concrete, Inc. By production line precasting techniques forming of the structure is eliminated with a resultant saving in time and construction costs. The process is now in use on some Federal government projects.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Strain measurement devices

For use with wire resistance strain gages a strain meter with built-in zero-balance potentiometers is said to accurately and rapidly indicate static strains at as many as 24 locations. A bridge balancer also is offered which can be used for both static and dynamic strain measurements.

For further information on the devices write Arthur R. Anderson, Springdale, Conn.

New type of gun for cement

A new type of gun for projecting cement, pumice and sand has been announced by Conair Sales, Inc., North Hollywood, Calif. The gun is a combination of both cement mixer and gun in one mobile unit. The water is added to the material through a metering device which is coupled with the speed of flow of the material so that the inspector can determine exactly the amount of water per cu yd or per sack of cement in the mix. The operational crew for the unit consists of a nozzle man, compressor engineer and a rod man.

Further information can be obtained from Conair Sales, Inc., 12147 Riverside Drive, North Hollywood, Calif.

Steel scaffolds

How to put workers at job level—safely, is the subject of a new 4-page Bulletin PSS-14 on "Trouble-Saver" Sectional Steel Rolling Scaffolds. It describes and illustrates a variety of arrangements of prefabricated steel sections being used as movable scaffolds for repairing, painting, decorating, plastering and other maintenance work. Typical installations are shown, together with methods for assembling sectional units to adapt scaffolding to space, height and shape requirments of different jobs. Text covers erection advantages and safety features. Copies are available from Patent Scaffolding Co., 3821 12th Street, Long Island City 1, N. Y.

Recommended specifications for lathing, furring and plastering

The second edition of "Handbook of Recommended Specifications for Lathing, Furring and Plastering" has been released by the National Foundation for Lathing and Plastering, Inc. The 40-page booklet includes sections on materials, grounds, application of lath, partition details, furred walls, ceilings, beams and cornices, proportions, mixing, application of base coats and applications of finish coats. An appendix includes structural support details, proportioning of aggregates, ventilation and frost protection, plastering on concrete and painting over new plaster. Hints

are also given to specification writers on furring, lathing and plastering.

Copies are available at 50 cents each from the National Foundation for Lathing and Plastering, Inc., Madison-LaSalle Bldg., 173 W. Madison St., Chicago 2, Ill.

Permite membrane curing compounds

A new bulletin describing Permite membrane concrete curing compounds has been published by Aluminum Industries, Inc., Cincinnati, Ohio. The bulletin announces the development of two compounds: Permite V-167, used for all horizontal concrete surfaces and for vertical surfaces below ground and Permite V-169, used for all vertical concrete surfaces where nondiscoloration is an important factor.

Description of technical features, methods of application and a testing laboratory report are included in the data.

Copies of the bulletin may be obtained by writing Aluminum Industries, Inc., 2438 Beekman Street, Cincinnati 25, Ohio.

Vermiculite floors and insulation

Specifications for vermiculite concrete floors and floor fill insulation have been released by the Vermiculite Institute. Included with the specifications are illustrated construction details of vermiculite concrete roof decks and roof insulation.

Copies can be obtained from the Vermiculite Institute, 208 South LaSalle Street, Chicago 4, Ill

Concrete test cylinder molds

In a 4-page leaflet, Bulletin No. 649, the Moline Iron Works illustrates the use of concrete test cylinder molds.

Further information can be obtained from Moline Iron Works, Moline, Ill.

Crack sealer

Arno-Cord, a tough 3-ply ¼-in. cord saturated with a moistureproof wax compound, has been released to the market to be used in sealing cracks around windows, etc. The cord is said to be self-sealing, nondrying and permanently pliable. The manufacturer claims that it can be applied, removed for window cleaning or ventilation and reapplied again.

Information on the new product can be obtained from Arnaud Corp., 17 John St., New York 7, N. Y.

Air entrainment

"Facts on Modern Placement of Concrete through Air Entrainment," a 32-page booklet released by Autolene Lubricants Co. is a digest of questions and answers on air entrainment. The booklet covers various phases of the use of air-entraining agent Protex AEA through 67 questions and answers.

Copies can be obtained from Autolene Lubricants Co., Industrial and Research Division, 1331 West Evans Ave., Denver 9, Colo.

Form-tying devices

Descriptions, construction details and engineering data are presented on form-tying devices and accessories in the latest Richmond Screw Anchor Co., Inc. catalog. The publication has a section devoted to charts and tables on the design of formwork, including charts on form pressures, basic data for wood form design, form deflections and allowable loads.

Copies may be obtained from Richmond Screw Anchor Co., Inc., 816-838 Liberty Ave., Brooklyn 8, N. Y.

Load-transfer assemblies

A 14-page booklet released by Richmond Screw Anchor Co., Inc., describes and illustrates the use of load-transfer assemblies in concrete pavement joints. Construction details are given for contraction and expansion joints. The action of concrete pavements is discussed as well as the controlling factors in load-transfer assemblies.

Copies of the booklet are available from Richmond Screw Anchor Co., Inc., 816-838 Liberty Ave., Brooklyn 8, N. Y.

Soil-cement roads

The Portland Cement Assn. has released a revised "Soil Cement Roads Construction Handbook." The 93-page booklet is devoted to the details of building soil-cement pavements, including the laying out of the project, laboratory tests, equipment and procedures and information required for obtaining control factors during construction.

Most of the material deals with soil-cement paving as used for road construction but several chapters deal with airport and other paving uses. Copies of the booklet may be obtained from district offices of the Portland Cement Assn.

Concrete masonry farm houses

In addition to typical house plans this 12-page pamphlet contains illustrations of good construction practices and exterior finishes for concrete farm homes. It is available at district offices of the Portland Cement Assn.

New shore clamp

New shore clamps, claimed by the developers to cut shoring costs in half, may be used with 4 x 4's or paired 2 x 4's. The manufacturers say that these clamps will safely support 6000 lbs and can be used for any temporary structure such as ramps and scaffolding.

For complete details of this shoring system, write Dept. 027, Farmers Tool and Supply Co., 4615 Washington St., Denver 16, Colo.

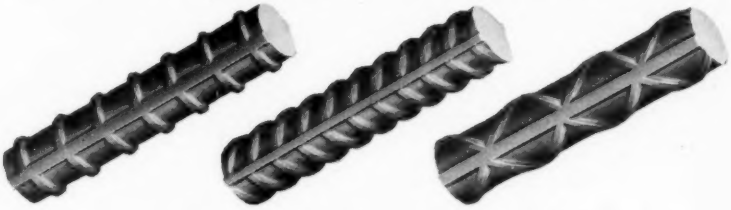
Magnesium forms

A concrete form made of magnesium, taking advantage of the light weight, dimensional stability, and lack of corrosion of this metal is soon to be available.

Further information on these forms may be obtained from Symons Clamp and Mfg. Co., 4249 Diversey Ave., Chicago 39, Ill.

THIS IS 1950!!!

DESIGN FOR HIGH TENSILE STRENGTH RAIL STEEL WITH THE NEW SPECIAL ANCHORAGE DEFORMATIONS



ASTM-A305-49 DIMENSIONAL REQUIREMENTS

Number of Bar	Wt. per foot	Nominal Dimensions*			Minimum* Height of Lugs
		Diam.	Area	Perimeter	
3	.376	0.375	0.11	1.178	0.015
4	.668	0.500	0.20	1.571	0.020
5	1.043	0.625	0.31	1.963	0.028
6	1.502	0.750	0.44	2.356	0.038
7	2.044	0.875	0.60	2.749	0.044
8	2.670	1.000	0.79	3.142	0.050
9	3.400	1.128	1.00	3.544	0.056
10	4.303	1.270	1.27	3.990	0.064
11	5.313	1.410	1.56	4.430	0.071

*Note all dimensions are in inches

Specifying Rail Steel reinforcing bars to ASTM A16-35 insures average elastic limit of 65,000 psi. Take advantage of high strength steel by using high design stresses,—up to 30,000 psi.

Eliminate Hooks by specifying deformations to meet A305-49 thereby obtaining more than double former bond strengths and considerable reduction in bending and placing costs.

Achieve crack control by building with concrete bars having "Special Anchorage" rolled into every inch of length. The new closely spaced high deformations of A305, A16 bars prevent the microscopic cracks in concrete from accumulating into visible cracks in regions of maximum tension. The high elastic limit of Rail Steel prevents unusual elongations under unpredictable overloads.

Get the full story of Rail Steel in the colored sound film "Rail Steel in the World of Today" available without cost. Write for application blank to:

THE RAIL STEEL BAR ASSOCIATION
38 South Dearborn, Chicago 3, Ill.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 21 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES (ACI 616-49) 46-1

Price 50 cents (in special covers).

REPORT of COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

Supersedes 38-30 and 45-18.

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES.....46-2

Price 35 cents.

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS ...46-3

Price 35 cents.

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION 46-4

Price 35 cents.

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING 46-5

Price 35 cents.

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION..46-6

Price 35 cents.

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION.....46-7

Price 35 cents.

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES...46-8

Price 35 cents.

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

The use of these methods results in high strength monolithic construction having exterior and interior finished surfaces, with insulation incorporated into the construction, if desired.

NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM.....46-9

Price 35 cents.

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspection after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

Price 35 cents.

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

BOND OF CONCRETE REINFORCING BARS.....46-11

Price 35 cents.

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

PERLITE AGGREGATE: ITS PROPERTIES AND USES.....46-12

Price 35 cents.

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

THIN WALL CONCRETE SHIP CONSTRUCTION.....46-13

Price 35 cents.

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of ¾-in. and 1½-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

SPECIFICATIONS SHOULD BE REALISTIC.....46-14

Price 35 cents.

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4", multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

INSPECTION AND TESTING OF MATERIALS.....46-15

Price 35 cents.

NICOLAAS T. F. STADTFELD—Dec. 1949, pp. 237-248 (V. 46)

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

FLEXURE OF CELLULAR SHELLS...46-16

Price 35 cents.

F. E. WOLOSEWICK—Dec. 1949, pp. 249-256 (V. 46

After a brief discussion of the uses of cellular shells and methods available for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

CEMENT PERFORMANCE IN CONCRETE EXPOSED TO SULFATE SOILS.....46-17

Price 35 cents.

L. A. DAHL—Dec. 1949, pp. 257-272 (V. 46)

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

EARLY STRENGTH OF CONCRETE AS AFFECTED BY STEAM CURING TEMPERATURES.....46-18

Price 35 cents.

JOSEPH J. SHIDELER and WILBUR H. CHAMBERLIN—Dec. 1949, pp. 273-284 (V. 46)

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temperatures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

A SHORTCUT FOR DETERMINING REINFORCEMENT IN REINFORCED CONCRETE.....46-19

Price 35 cents.

V. BOGVAD-CHRISTENSEN—Dec. 1949, pp. 285-292 (V. 46)

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

PRECAST UNITS FOR SHORT-SPAN BRIDGES.....46-20

Price 35 cents.

ROBERT C. HANCKEL—Jan. 1950, pp. 317-328 (V. 46)

Bridge replacement in Lowell, Mass., where minimum traffic interruption was necessary, led to the adoption of precast reinforced concrete units. Available mobile hoisting equipment limited the maximum weight of sections to about 12 tons which permitted small bridges to be precast as complete structures, while for larger bridges, subassemblies were precast and incorporated into the bridge by a cast-in-place deck. The precasting procedures and construction processes are described and illustrated.

INFLUENCE OF SUBGRADES AND BASES ON DESIGN OF RIGID PAVEMENTS.....46-21

Price 35 cents.

KENNETH B. WOODS—Jan. 1950, pp. 329-348 (V. 46)

This paper has been developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions, and traffic volumes and loads are evaluated.

INSPECTION OF MASS AND RELATED CONCRETE CONSTRUCTION.....46-22

Price 35 cents.

LEWIS H. TUTHILL—Jan. 1950, pp. 349-360 (V. 46)

This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective: a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

EXPERIMENTAL GROUTING INVESTIGATION FOR CHIEF JOSEPH DAM.....46-23

Price 35 cents.

J. M. WELLS—Jan. 1950, pp. 361-376 (V. 46)

This paper describes the development of a method for the control of seepage into the excavation areas of the right abutment, and through the right abutment, at the Chief Joseph Dam project on the upper Columbia River in the State of Washington.

Laboratory studies are presented on various types of grout mixtures, as well as techniques and procedures for drilling in coarse gravel by jetting methods, procedure for grouting pervious gravels and the efficacy of grouting treatment.

USE OF POZZOLANS IN CONCRETE.....46-24

Price 35 cents.

RAYMOND E. DAVIS—Jan. 1950, pp. 377-384 (V. 46)

After stating the characteristics of pozzolonic materials, the effects of replacing with pozzolans part of the portland cement in concrete are considered briefly. Results of tests with fly ash and superfine diatomite are cited to show the possible use of these materials in the East and Midwest where natural pozzolans of the West are not economically available.

CONSTRUCTION OF LONG-SPAN CONCRETE ARCH HANGAR AT LIMESTONE AIR FORCE BASE.....46-25

Price 35 cents.

JOHN E. ALLEN—Feb. 1950, pp. 405-416 (V. 46)

The reinforced concrete arch hangar at Limestone Air Force Base is noteworthy for its size, being one of the largest of this type yet constructed in this country. The superstructure is composed of a thin reinforced arch shell, shaped like an inverted catenary. Construction procedures are described.

VOLUME CHANGES IN SMALL CONCRETE CYLINDERS DURING FREEZING AND THAWING.....46-26

Price 35 cents.

RUDOLPH C. VALORE, Jr.—Feb. 1950, pp. 417-436 (V. 46)

The volume-temperature behavior of small concrete cylinders was observed, using a new mercury-displacement dilatometer, during freezing and thawing cycles in which the range 40 to −20 F was traversed at various rates. Specimens cast from a mix containing 6 bags of portland cement per cu yd, plain and modified by the addition of an air-entraining agent, were tested in air-dry, vacuum-saturated and partially saturated conditions.

Volume-temperature relationships for air-dry specimens yielded uniform thermal expansion data, but those for vacuum-saturated specimens showed departures in the

form of transient expansions during freezing, and residual expansions following thawing of the order of 0.8 and 0.4 percent, respectively, a single slow cycle produced decreases in dynamic E exceeding 60 percent.
Much smaller departures were observed for partially saturated specimens (65 to 85 percent of vacuum-saturation) and included, in addition to transient and residual expansions, secondary effects of freezing termed "shrinkage" and relaxation phenomena. The magnitude of the transient and residual expansions appeared to depend upon the rate of cooling and heating, the degree of saturation of the specimen, and, during the slow cycle, upon the degree of supercooling before freezing.

INSPECTION AND CONTROL OF CONCRETE FOR HIGHWAY AND BRIDGE CONSTRUCTION.....46-27

Price 35 cents.

H. W. RUSSELL—Feb. 1950, pp. 437-444 (V. 46)

Inspection procedures and personnel assignments used by the Illinois Division of Highways for large, medium and small jobs are described. The importance of assurance that materials used are adequately tested and approved before shipment, determination of proper proportions by trial mixtures, control of air entrainment, correction of batch weights and proper preparation and testing of concrete specimens are emphasized. The inspection of ready-mixed concrete supplied to small jobs poses a problem still lacking an adequate solution.

SOME APPLICATIONS OF ELECTRIC SR-4 GAGES IN REINFORCED CONCRETE RESEARCH.....46-28

Price 35 cents.

EIVIND HOGNESTAD and IVAN M. VIEST—Feb. 1950, pp. 445-456 (V. 46)

The writers describe some applications of SR-4 gages in reinforced concrete research at the University of Illinois. Special emphasis is given to the problems involved in waterproofing gages attached to steel which is later embedded in concrete.

THE ULTIMATE STRENGTH OF REINFORCED CONCRETE BEAMS...46-29

Price 35 cents.

S. D. LASH and J. W. BRISON—Feb. 1950, pp. 457-472 (V. 46)

The results of tests to failure on 57 small reinforced concrete beams are presented, and it is shown that they can be explained satisfactorily on the basis of a simple plastic theory of design. Formulas are proposed for determining the moments of resistance and the critical percentages of reinforcement.

A WAY TO BETTER PAVEMENT CONCRETE.....46-30

Price 35 cents.

F. H. JACKSON—Mar. 1950, pp. 489-496 (V. 46)

This paper discusses the performance requirements of concrete pavements from the standpoint, primarily, of the quality of the concrete as a material. It is pointed out that the lack of durability of much of our present day concrete may be due to our methods of construction. The writer believes pavement durability could be significantly improved by using a scientifically proportioned mix of dry consistency, well compacted by vibration or tamping, and with close control over aggregate gradation, in place of the oversanded plastic mixtures which we now use. He does not believe that air entrainment is necessarily the final answer to the problem of surface deterioration but that we should seriously consider overhauling our entire construction practice as well as our present methods of controlling the uniformity of aggregate gradations.
Current practice in the construction of concrete pavements and airport runways in Great Britain and past practice in Germany in the construction of the autobahnen are discussed to show that the placing of harsh, dry mixtures with close control of aggregate gradations is entirely feasible from the construction standpoint.

DESIGN AND CONSTRUCTION OF A CIRCULATING WATER INTAKE..46-31

Price 35 cents.

W. S. COLBY—Mar. 1950, pp. 497-508 (V. 46)

The author describes the design and construction of a 250,000 gpm power plant intake. An interesting feature of the pump well cofferdam was the reinforced concrete ring wales, which resulted in savings in operations because the interior of the cofferdam was kept clear for excavating, driving of drilled-in caissons, tremie operations and placing of the pump well concrete.

THE CONTRACTOR'S VIEWPOINT OF INSPECTION.....46-32

Price 35 cents.

DONALD C. ANDREWS and NOMER GRAY—Mar. 1950, pp. 509-512 (V. 46)

The interests of owner and contractor are compared and recommendations are made on specifications, personal qualifications of the inspector, and methods of minimizing friction with the contractor.

CORROSION PROTECTION OF THIN PRECAST CONCRETE SECTIONS.....46-33

Price 35 cents.

D. H. PLETTA, E. F. MASSIE and H. S. ROBINS—Mar. 1950, pp. 513-528 (V. 46)

A new electrical resistance technique for measuring the rate of corrosion of steel reinforcing is described. The method employs a thin ribbon 0.008 x 0.25 in. as the resistance element embedded in 6 x 12 in. thin panels, and a Kelvin bridge sensitive to 0.0001 ohm. The data plotted in dimensionless parameter form enable the *half-life* of concrete to be determined at a comparatively early age. The term half-life is defined as the time required for the cross-sectional area of the reinforcing to decrease by one-half its original value due to corrosion. Six mixes, three water-cement ratios, four exposure conditions and three depths of cover were examined.

DESIGN OF HEXAGONAL BINS...46-34

Price 35 cents.

PAUL ROGERS—Mar. 1950, pp. 529-540 (V. 46)

The analysis of hexagonal bins is presented with a practical example, showing the details of computations for lateral pressures, inward pull, hanging loads, diaphragm action and the design of the walls, beams and columns.

PRECAST CONCRETE IN BRITAIN..46-35

Price 35 cents.

P. G. BOWIE and A. R. COLLINS—Mar. 1950, pp. 541-556 (V. 46)

Precast frames for airplane hangars, garages and farm buildings are discussed. Several systems using precast units in housing construction are described and illustrated. Prestressed precast railroad ties and transmission-line poles are mentioned briefly.

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"Knowledge changes you say; Ah no; knowledge does not change; we change."

The Zig-Zag Course of Concrete Progress*

By HERBERT J. GILKEY†

SYNOPSIS

Retiring ACI President Gilkey emphasizes that concrete has not become more complex, but rather that as knowledge advances, usually in a zig-zag course, not a straight line, more problems and seemingly contradictory half-truths become evident.

At the west end of the Chicago Midway, about 113 blocks or 14 miles south of us, stands the 28-year old Lorado Taft-John Earley monument in concrete,‡ the Fountain of Time. About 100 human figures are shown as successive waves passing in review for a length of 120 ft before the large, detached, centrally-located figure of Father Time. Taft's concept came from a couplet written by Henry Austin Dobson: "Time goes you say; Ah no, Time stays; we go."

This couplet can be appropriated almost bodily to express a thought parallel if not quite identical: "Knowledge changes you say; Ah no, knowledge does not change; we change."

Just as walking is but a succession of interrupted falls, the entire history of human progress is a succession of stumbling half-truths and misinterpreted facts. The "accepted facts" of today become the "recognized fallacies" of tomorrow.

Of course, the facts haven't changed; it is simply a case of "another side of the elephant heard from." We blind men grope in the belief that some day we will get all the way around that elephant and synthesize our findings; a happy, stimulating thought until we ask, "What then?" No unanswered questions; no mystifying inexplicables; no disagreements; the end—what lies beyond the end? Perfection; complacency; stagnation; certainly not ACI.

*Presented at the ACI 46th annual convention, Chicago, Ill., February 21, 1950. Title No. 46-36 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 8, Apr. 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Professor and Head, Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Ia.

‡*Proc.* American Concrete Institute, V. 19, 1923, pp. 178-190.

With the hydra-headed problems of concrete, however; with every answer posing two bigger and better questions, our continued groping and the future of ACI seem assured.

We mix and we mold; we fumble and flounder. A discerning bacteriologist observed that the nose harbored more germs than any other portion of the body and it was not until all noses had been amputated that he discovered further that the nose functions as the intake strainer or trash rack for the body. We've made similar mistakes with our concrete.

People long for the good old days when concrete was simple. Concrete was never simple; we were. Or possibly concrete was always simple and, in the words of the hymn: "On that bright and happy day we shall know our *concrete* better, when the mists have rolled away." Perhaps our current thinking is merely snarled and needlessly complicated; in terms of army slang: "SNAFU" (situation normal, all fouled up).

The zig-zag course of concrete progress differs in no essential respect from that of medical or any other form of progress. Winding trails are the exploratory forerunners of broad highways and Roman Roads. There are dozens of seemingly incompatible half-truths, which, when understood, are not at all contradictory. For example:

1. To produce strong, durable concrete:
   (a) When in doubt use more cement.
   (b) Use just as little cement as possible.
2. (a) Tamp concrete vigorously.
   (b) Don't overwork the concrete.
3. (a) Don't let temperatures get too high after placement.
   (b) High-pressure steam curing at 300 or 400 degrees produces high-quality early-strength concrete.
4. (a) For a tall structure, sliding forms and continuous pouring is best.
   (b) Limit the height of pour in any one operation to five feet (or other relatively low value), letting days elapse between lifts.
5. (a) Winter is the hazardous season in the placement of outdoor concrete and warm-weather concrete hardens much faster than cool-weather concrete.
   (b) Warm-weather concrete cracks worse than cool-weather concrete.
6. (a) Concrete should have as little mixing water as is compatible with proper placement.
   (b) Concrete should be kept saturated for curing.
   (c) Concrete is soluble in water.
   (d) Air-dried concrete is stronger than the same concrete resoaked.
   (e) Oven-dried concrete is weaker than air-dried concrete.
7. (a) Concrete frozen soon after placement may suffer little ultimate damage (if subsequently allowed to cure at suitable temperatures).
   (b) Concrete frozen a day or so after placement may be seriously damaged.
   (c) Hardened concrete is likely to be undamaged by a cycle or two of freezing and thawing.

(d) Hardened concrete, if saturated, may be severely damaged by a cycle or two of freezing and thawing.

8. (a) Reactive aggregate may be expected to produce serious deterioration.
   (b) A special reactive admixture is being used in the Hungry Horse Dam—third highest in the world.
9. (a) Reinforcement strengthens concrete.
   (b) Reinforcement frequently promotes cracking and deterioration—makes the concrete fight with itself.
10. (a) The strength of the concrete is a function of the ratio of cement to aggregate.
    (b) The strength of concrete is solely a function of the ratio of cement to water.
    (c) The strength of concrete is solely a function of the ratio of cement to voids.
11. (a) For high-strength, durable concrete supply, after placement, ample curing water; don't let the concrete dry out.
    (b) For high-strength durable concrete extract, after placement, as much of the mixing water as is possible by use of vacuum mats, porous molds, or absorbent form lining.
12. (a) For high-strength durable concrete keep the voids at a minimum by using the least amount of mixing water compatible with proper placement.
    (b) For workable, durable concrete introduce four to six percent of voids by the use of air-entraining cement or air-entraining admixtures.
13. (a) A few years ago we used vacuum mixers.
    (b) Now we use air entrainment.
14. (a) A neat cement requires about 20 percent of its weight of water for complete hydration. For concrete, workability requirements necessitate a ratio of water to cement of 50 percent and upwards; two to four times enough for complete hydration.
    (b) Why then may even complete retention of the mixing water be insufficient fully to meet the hydration or curing need?
    (c) If plenty of water is so vital for proper curing, why not add it as extra water when the concrete is mixed?
15. (a) Alternate wetting and drying is essentially a weathering process and is damaging to concrete.
    (b) Alternate wetting and drying is often highly beneficial to concrete.
16. (a) Adequate initial curing is especially important for concrete subject to severe outdoor exposure.
    (b) Adequate initial curing is especially important for protected interior concrete subject to no exposure.
17. (a) Much concrete is insufficiently mixed.
    (b) Overmixing may be harmful.
18. (a) At constant water-cement ratio the concrete is weaker than its mortar.
    (b) At constant water-cement ratio and slump concrete is stronger than mortar.

Such apparent contradictions could be continued indefinitely. No actual contradiction, however, is involved in any set of statements made. All one needs to do to rationalize and harmonize them is to see just a little farther around that elephant.

Atomic laws, while but recently recognized, are not of recent enactment; the same applies to concrete. Alkali deterioration, the problems of mass concrete, and a need for air entrainment existed long before they were recognized.

Since the present purpose is neither to provoke controversy nor to dispense information, only illustratively shall I attempt to point out the lack of actual incompatibility—and that for but one limited area. In the interest of essential brevity, my statements will be sketchy, assertive, and but meagerly guarded with qualifying pros and cons. Let us remain aware that the central theme revolves not around the truth or falsity of what I say on the topic selected; that is irrelevant. My theme song is that seeming contradictions are not insolvable mysteries but are simply proof conclusive that we are still in the groping, study stage. If you disagree with what I have to say regarding the aspects discussed: cement vs. aggregate vs. water vs. voids vs. air entrainment, skip it; select your own seeming contradiction and start there—there are plenty from which to choose.

One needs but refer to the First Joint Committee Report (1916), or to other of the voluminous concrete literature, current even then (some of you need but refer to your own memories), to realize that the then-criterion of concrete strength and excellence was the ratio of cement to aggregate; 1:2:4 was standard at about 2000 psi, 1:3:6 was lean, 1:1.5:3 was rich. There was no slump test and the water content, if mentioned at all, was couched in general terms of such mixture characteristics as easily workable, stiff, or fluid. While a bit crude the aggregate ratio wasn't a bad criterion, especially when comparisons were between mixtures from the same or similar aggregates and of comparable workabilities. Strength and stiffness (which determined the value of n in reinforced concrete design) did vary about as the ratio of cement to aggregate. The richer concretes were also the more durable.

Then, in 1918, Abrams came forth with his classic Lewis Institute *Bulletin* No. 1 to demonstrate conclusively that the amount of the aggregate has nothing (or at most, little) to do with the strength of the hardened concrete; it was the ratio of cement to water expressed by Abrams (and thereby established) as an inverse ratio. Water, heretofore generally ignored, became dominant and the aggregate ratio heretofore dominant went out the window—or did it?

We of this group long since came to recognize that the water-cement ratio criterion was a refinement, not a reversal of the aggregate-cement ratio criterion. It is only the waiters and the women present in this room who might not now realize that at comparable workabilities and gradings, more aggregate requires more water and, except for ease of measurement and control, one criterion is about as good as the other.

In France, long prior to the work of Abrams, Feret had evolved a voids-cement ratio criterion, as did also Talbot and Richart in their Illinois *Bulletin* 137 of 1923, five years after Abrams' water-cement ratio pronouncement. Once again to the layman (and to many others who considered themselves to be more than laymen), there was confusion. When theories clash and authorities disagree, wherein lies truth? It is now universally known what was then understood by few, that there was no clash. The voids-cement ratio (or cement-space ratio, another version of the same) proved to be a refinement of the water-cement ratio criterion. As a workable, well-established procedure for the design and control of mixtures, the water-cement ratio, trial-batch procedure remains dominant. The techniques of application differ widely but since, in workable non-air-entrained concrete, the total voids (air plus free water as mixed) are 85 to 90 percent water, the essential end-point difference between water-cement ratio and voids-cement ratio is not great.

Up to now this has been old stuff with which even the least experienced of you is familiar but now we come to that revolutionary development of the current decade, "air entrainment." Surely we have here some real incompatibles. The benefits from air entrainment have been so conclusively demonstrated as to leave little room for question as to its efficacy, but in the minds of most of you there are probably a lot of unanswered questions and considerable skepticism about some of what we previously thought we knew. To whatever extent you have attempted to think the thing through, these are some of the questions which you have doubtless been pondering or avoiding:

1. Air is voids; increasing the voids lowers the strength; lower strength concrete is less durable concrete but *air-entrained concrete is more durable.* Why?
2. Water is voids; increasing the mixing water increases the voids; increasing the voids lowers the strength; *this lower strength concrete is less durable.* True.

Are we to conclude then that air voids and water voids both weaken concrete but that air voids increase its durability and that water voids decrease it? The answer is "yes" and "no."

Suppose, for example, that we take a high water-cement ratio concrete, one with relatively high water voids. After the concrete has hardened, suppose we let it dry out in air (or otherwise). A sizable increment of the free or capillary water (especially in the larger capillaries) evaporates and what was a high water-void concrete, now becomes a high air-void concrete. Does this concrete now have the durability of an air-entrained concrete having a comparable volume of air voids from the outset? The answer is "no" and the question is "why?"*

*This answer has to be qualified. If frozen and thawed in the partially dried-out state the durability would have been appreciably greater than when saturated—see Powers, ACI JOURNAL, June 1947, *Proc.* V. 43, pp. 1089-94—but this is a different, though not unrelated, aspect of the matter.

The answer appears to be that the significant factor as regards durability in relation to air voids is in their nature. If the air voids are in the form of passageways such as those formerly occupied by water, the water can re-enter and freeze, or bring with it deleterious substances, or percolate on through taking into solution essential constituents of the concrete. If the air voids are small, closed bubbles like those introduced by air-entraining agents, the concrete not only remains impervious to the entrance or passage of water, but it has an induced springiness from numerous little air-domes, like the air-dome on a pump, which absorb, without damage, much of the expansive pressure induced by freezing water or crystallizing salts.*

Under item (1) we left the concrete weaker but more durable by virtue of the increase in air voids. In any concrete mixture, much more water is required to lubricate the mixture and make it workable than is required for hydrating the cement, especially in the early stages of curing. Air, like water, is a fluid and, like water, it too provides lubrication for the mixture. Initially the only thought was to add air, thereby gaining increased durability at the expense of some reduction in strength. The added fluidity or mobility, due to the added air, was soon recognized, however, and without loss of placing quality it is possible to omit enough of the mixing water to compensate largely for the weakening effect of the added air voids. Usually even with a redesigned mixture, however, there is still some reduction in strength but this need not be great and sometimes there is a slight increase. While air voids and water voids affect strength similarly, the effects for a given voids content are not identical—this has long been known.

The facts are essentially as stated; some of you may differ with opinions expressed or explanations volunteered—that is a small matter of slight consequence if we can be agreed that: aggregate-cement ratio, water-cement ratio, voids-cement ratio, and air entrainment involve no contradictions of evidence. "Knowledge changes you say; Ah no, knowledge does not change; we change."

Yes, concrete used to be simple, whether it was in the days of the Romans, the reign of T. Roosevelt or the era of Coolidge; so much more can be wrong with it now than heretofore (things for which not even FDR or HST can validly be blamed). In the good old days there wasn't any flu, only grippe; appendicitis and ulcers were mere stomach aches; in spite of everything some people lived to be very old. As we view and admire enduring remnants of the past, in comparison with a Parker

*It seems conclusively to have been proved that the air entrainment phenomenon is strictly physical—not chemical. Moreover, it has been further demonstrated that the size of the sand particles is an important factor in determining how effective an air-entraining agent will be. Air bubbles of suitable size (25 to 45 microns) and amounts (4 to 6 percent by volume of the concrete) are best formed as clusters around particles of sand that lie within the 30 to 100 mesh range. Neither the very fine nor the relatively coarse particles contribute much to air entrainment possibilities.

Dam or an ailing highway, might we not introspectively ask ourselves a few such questions as:

How did the traffic, or other service and environmental conditions to which the parent structure was subject, compare with those for its modern counterpart in the northern United States or Canada?

For this remnant how much other concrete of similar vintage was there of which there remains no visible trace? Darwinian law, "Survival of the Fittest," applies to concrete as surely as it does to crustacea.

Then there was a period when we were sensitive about our concrete; we didn't like to play up its frailties. Once having adopted the slogan "Drink Hood's buttermilk and live forever," we objected to publicizing the funerals of our buttermilk drinking constituency—ditto "Concrete for Permanence." Now, though the slogan is retained, we have swung to the other extreme until the sins and shortcomings of concrete are placarded and publicized with the fervor of an old-fashioned revival, or a Russian courtroom. Sometimes, in the words of the colored minister, our present-day candor is more like "braggin' than confessin'." The long-time durability tests under P.C.A. support and sponsorship constitutes what may well be the most broad gage attempt ever made by an entire industry to discover what is wrong and how to remedy it.

In this gathering we have a heterogeneous mass of human aggregate, clean (I hope) and well-graded—perhaps inert; individual fragments from many sources: contractors and salesmen, producers and consumers; engineers and architects; researchists and educators, all literally bound together by a paste of portland cement. If concrete had no unsolved problems, I would have no audience; you wouldn't be here and neither would I.

Thanks to the existence of bull-headed, venturesome, nonperfectionist souls with sufficient vision to recognize certain unique qualities in concrete, but blind enough or stubborn enough not to see or be deterred by its manifold shortcomings, concrete long ago took an important place in the highly competitive realm of construction materials. Until it is superseded by something that can perform its functions better, it will continue to hold a position of great structural importance. Concrete owes its important position among structural materials to the fact that, in spite of recognized faults and many unsolved problems, it is economical, highly resistant to fire, wind, water, earthquakes and war, and lends itself readily to almost any type of decorative or architectural expression, ranging from the conservative to the fantastic.

The P.C.A., the materials man and the builder may expect to carry on as long as there is concrete but only so long as concrete remains imperfect can there be an ACI. For us 'tis the fascination of the chase; we are here solely out of curiosity; to find answers to questions; solutions to

problems. If the time ever comes when we know the answers, *concrete* continues but *we* are through. It is normal and proper that we should bemoan the slowness with which we seem to progress; that at times we should even have attacks of acute nostalgia for the good old days when our *concrete* seemed less *abstract.*

Admittedly the course of concrete has been a zig-zag affair; not all motion has been progress; we change the constituents of our cement, the fineness of the grinding; we add water and we take it away; we move ahead and we back track. It is only with the degree of detachment supplied by the calendar that we are able sometimes to harmonize seeming contradictions and convince ourselves that the trend has been forward; that we are getting further around that elephant. Nevertheless, as stated earlier, the questions of concrete continue to be hydra-headed. An eminent medical professor once addressed his graduating class about as follows: "While I have taught you to the best of my ability, I feel impelled to make a confession: half of what I have taught you is false. Unfortunately I don't know which half."

For 45 years the ACI has pursued its zig-zag course of plodding concrete progress. Some of us would view those years with pride of accomplishment; others with chagrin that the trail should have been so winding. Extrapolation from the past seems to indicate that insofar as the future is controlled by the amount of work yet undone, to that extent the future of the Institute is assured. The next 45 years promises to be busier than the last and the course will probably be just as winding. After all its more fun that way—isn't it?

Discussion of a paper by Herbert J. Gilkey:

The Zig-Zag Course of Concrete Progress*

By M. SPINDEL†

It is good to have a retrospect on the ups and downs of concrete progress given from time to time by a prominent expert. The author explained why this progress was not and could not be a "straight line" but rather a "zig-zag course," and illustrated the many apparent contradictions which were seemingly incompatible half-truths, which, when understood, are not at all contradictory.

Was this rather strange zig-zag course with its half-truths and misunderstandings really necessary and unavoidable, if compared with progress in the development of other engineering materials, such as metals and plastics? The casualties of the course under discussion were perhaps even greater than those of medical progress.

Was this course not partly due to some previous tendency to hide the fact that cement as a powder, and still more as a paste, during its manufacture, proportioning, mixing, placing and curing was already *per se* a complex matter, a complexity which was increased greatly by the numerous hazards arising from the various sands and coarse aggregates?

Is it not a fact that, although this tendency to hide the complexity of cement and concrete problems exists no longer, even the approximate composition, *i.e.*, the percentages of cement, aggregate, water and air, of any concrete is still unknown to most of the makers in the field and even to some in laboratories?

As a rule it is known how much sand and coarse aggregate has to be or is added by volume (*i.e.*, including the space taken by air and moisture) to a bag of cement and how many gallons of water has to be or is added to a bag of cement. From this the percentages of the absolute volumes of cement, aggregates, water and air per unit of fresh concrete have to be calculated or guessed if the fundamentals are to be known. As the writer pointed out in

*ACI JOURNAL, Apr. 1950, *Proc.* V. 46, p. 573. Disc. 46-36 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.
†London, England.

previous discussion,* this is the usual way, although according to ACI recommended practice the absolute volumes of the various constituents per unit of fresh concrete should be given so that the percentages can be calculated.

The author is right in saying that there are certain mathematical relations between the proportions of cement and aggregate, the water-cement ratio, the voids-cement ratio and the cement-space ratio which are the basis of various properties of concrete, such as density, water absorption and permeability, strength and resistance to weather and frost. These mathematical relations were shown by the writer in various publications since 1928 and referred to in various discussions in the ACI JOURNAL.

It is to be hoped that the excellent way in which the author dealt with this matter will encourage science and practice to consider and examine concrete more intensively, not only with regard to the percentages of the main components but also by microscopical examination. This is especially important with regard to the size, shape and distribution of air pores and their influence on the properties of concrete, as was pointed out by the writer in the discussion of a paper by Paul Klieger.†

Many contradictions in the course of concrete progress occurred because theory was not always in step with the requirements of practice. The latter could not always wait until the theoretical problems could be solved satisfactorily but had to take the lead as, for example, with admixtures to concrete. These were used with success at a time when theory had not yet shown them feasible. In the last ten years much has been achieved in favor of admixtures, but much remains to be done to make it clear to all concerned that admixtures to mortar and concrete are of similar importance with cement, aggregate and water. Admixtures have to be dealt with on the sound basis of theoretical and practical knowledge and experience to avoid a further zig-zag course in this important line of concrete progress.

If all suitable means are used, high-grade concrete can be made with qualities similar to those of the best natural stones. The advantages and superiority of plain and reinforced concrete and precast and prestressed concrete for modern structures must not be made dubious by any further zig-zags which could possibly be avoided.

*Discussion of the symposium "Entrained Air in Concrete," ACI JOURNAL, Dec. 1946, Part 2, *Proc.* V. 42, p. 604-1.

†Discussion of "Effect of Entrained Air on Concretes Made with So-Called 'Sand-Gravel' Aggregates," ACI JOURNAL, Dec. 1949, Part 2, *Proc.* V. 45, p. 164-1.

Unusual construction methods with high lifts applied to mass concrete are described in

Concreting on the Ottawa River Projects of the Hydro-Electric Power Commission of Ontario*

By A. L. MALCOLM† and R. B. YOUNG‡

SYNOPSIS

Construction procedures on three Hydro-Electric Power Commission of Ontario dam projects are described including the placing of concrete in lifts up to 50 ft, material and concrete handling systems, classes of concrete, sand blending and aggregate production, treatment of joints, concrete proportioning, winter concreting, and concrete control measures. A discussion of the merits of high lifts in mass concrete structures concludes the paper.

INTRODUCTION

The Hydro-Electric Power Commission of Ontario has under construction at present, five large hydro-electric power developments. Three of these, with an ultimate capacity of some 830,000 horsepower, are on the Ottawa River in eastern Ontario, and will require in their construction more than 1,400,000 cu yd of concrete. The methods followed in the distribution and placement of this concrete involve several unusual departures from normal practice for like work which are thought to be of sufficient interest to record. They are,

1. Lifts up to 50 ft in height.
2. Bailey bridging for trestles and some form supports.
3. Conveyor belts, trucks, concrete pumps and drop chutes in various combinations to distribute concrete.
4. Two classes of concrete in the dams, one for the face and the other for the core.
5. A combination of two sands blended at the batching plant.

*Presented at the ACI New England Regional Meeting, Boston, Mass., Nov. 10, 1949. Title No. 46-37 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 8, Apr. 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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The three Ottawa River developments, their location and size are,

Des Joachims
42 miles west of Pembroke, 8 units, 480,000 hp. Total concrete 880,000 cu yd. Started in 1946.

Chenaux
12 miles west of Renfrew, 8 units, 160,000 hp. Total concrete 225,000 cu yd. Started in 1947.

La Cave
6 miles above Mattawa, 6 units, 192,000 hp. Total concrete 370,000 cu yd. Started in 1948.

The concreting methods on the three jobs are basically the same and differ only in details brought about by differences in local conditions. For this reason, Des Joachims, the largest, will be described in some detail and the others discussed only in those particulars in which they differ from it.

RAPIDES DES JOACHIMS

The Des Joachims project comprises the construction of a powerhouse, and three dams, two large and one small. Comparative yardages are as follows:

	Concrete, cu yd
McConnell regulating dam 125 ft high, 1600 ft long	270,000
Main dam and headworks 160 ft high, 2600 ft long	475,000
Power house and high voltage switching 560 ft long	110,000
Auxiliary dam 40 ft high, 1200 ft long	25,000
Total concrete	880,000

The general layout of the development is shown in Fig. 1. The river will be raised about 100 ft at the main dam and this will divert water through the McConnell Lake gap to the regulating sluice dam. All flood flow and excess water over 40,000 cfs will pass through the lake, changing it to a flood discharge channel. To facilitate this diversion, 500,000 cu yd of earth and gravel have been excavated as a pilot channel from the foot of McConnell Lake to deep water in the main river below the village of Des Joachims. At the main dam, sixteen 19-ft head gates will control flow through 22-ft diameter steel penstocks to the eight 60,000 hp units in the powerhouse, the concrete foundations of which are integral with the main dam (Fig. 2). At the McConnell dam are

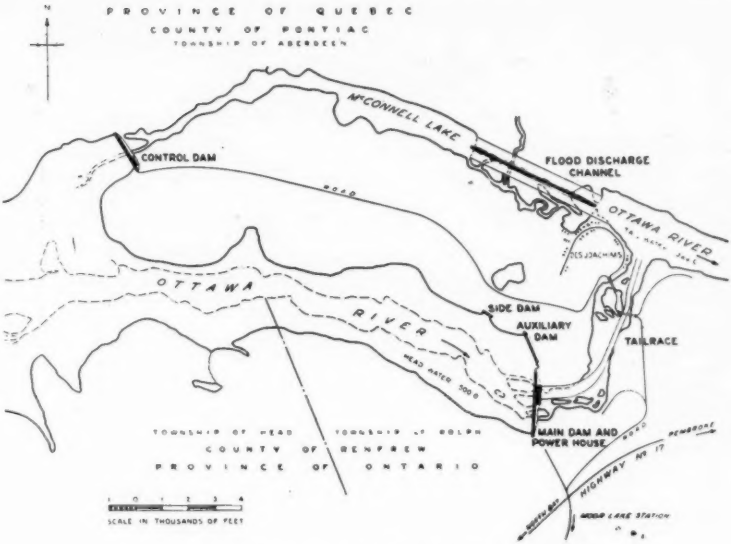

Fig 1—General arrangement of Des Joachim Development

forty 16-ft sluices fitted with stop logs and six 40-ft sluices equipped with electrically operated sluice gates for regulation and rapid flood control.

Use of Bailey bridge

At Des Joachims, the Commission first used modified Bailey bridge frames in the construction of various job structures, such as in the piers and trusses of the utility bridge built to support the two 30-in. belt

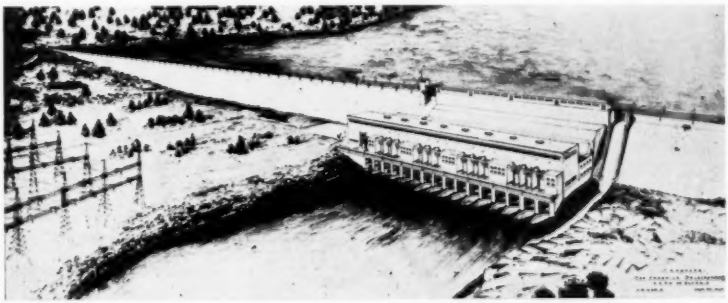

Fig. 2—Artist's conception of Main dam and powerhouse at Des Joachim when completed

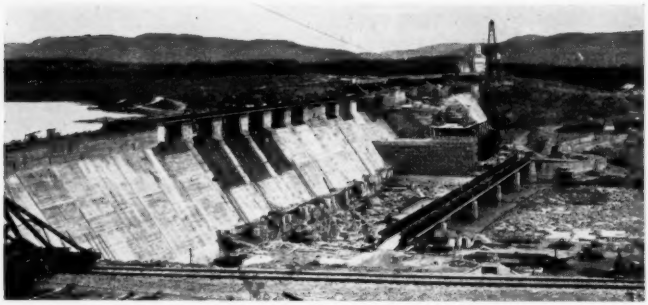

Fig. 3—Main dam nearing completion. Powerhouse substructure in and superstructure underway. Note Bailey bridge downstream from dam.

conveyors used to distribute concrete along the main dam. It was also used to support a conveyor belt for the stockpiling of aggregates, for much of the framework supporting the high forms of the main dam and entirely for the forms on the McConnell Lake dam. On the latter structure, the Bailey form cages were braced and strengthened to serve also as supporting piers for trusses carrying a belt conveyor and such accessory utilities as air, water and power lines.

The adaptation of standard 5 x 10-ft Bailey bridge panels for form construction introduced radical changes in form building. Whereas the usual practice is to back up 2 x 4-in. or 2 x 6-in. studding with 4 x 6-in. or double 2 x 6-in. waling strips, through which pass the ¾-in.

Fig. 4—Close-up of powerhouse area. Note Bailey trestle at right and above powerhouse which carries the belt conveyors used to distribute concrete.

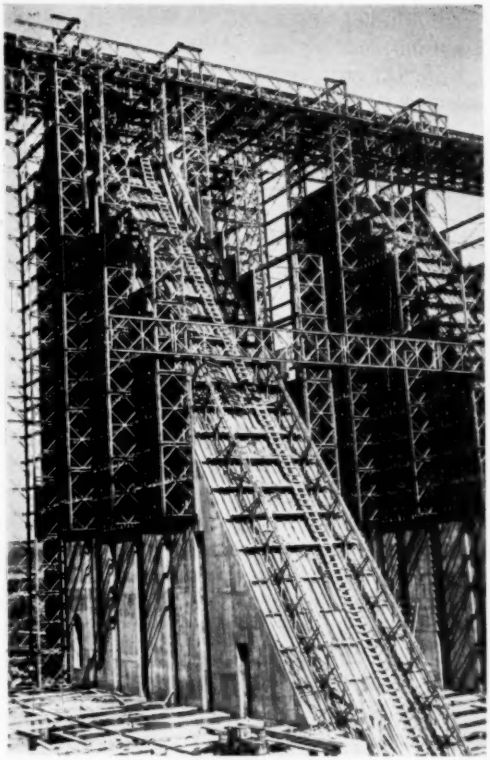

Fig. 5—Details of Bailey form construction

through bolts to hold the form together, the use of Bailey bridging eliminates the walers entirely. Fig. 3 to 5 show these bridge frames in place on the main dam. They are connected together on the ground (by 1½-in. tapered pins) then lifted into place by a caterpillar crane or cableway and stood vertically on special castings anchored to ¾-in. bolts set accurately in the footing of the dam. An initial or footing pour varying from six to ten feet high is always placed first to provide a level floor on which to work regardless of the type of forms used.

The vertical Bailey trusses are erected 17 ft 4 in. apart. On the inside face of these trusses, transom beams (I beams modified to suit Bailey design) are bolted horizontally to the trusses and vertically spaced at 5 ft centers. In this way the side of a Bailey form cage presents a grillage of 5 ft spaces, to which the 5 x 10-ft B. C. fir plywood panels are fastened with ½-in. bolts having a large flat head on the inside to

avoid crushing the wooden surface. The plywood panel is stiffened by 2 x 4-in. studding, the whole being mounted as a unit on the caging. To prevent spreading of the caging when placing concrete, the ¾-in. tension rods are carried through the forms from side to side—in the piers of Fig. 5, a distance of 20 ft; in the main dam, distances up to 40 ft. Some control has to be maintained on the rate of placing concrete in these as in other types of forms, the rate being limited to 2 ft per hour. This limitation becomes important only when concreting the top 50-ft lift as otherwise the area to be covered is so great that the rate of rise in the forms is limited by the capacity of the mixing plant.

Cement

Cement is received in bulk at a railroad siding about 2½ miles from the job and unloaded into closed trucks by a Fuller-Kenyon pump. Silos provide storage at the siding and mixing plant for some 45 carloads of cement. Cement can also be held in cars at the siding to augment this but as the mill supplying cement is on a direct railway line and the service is good, this has only been necessary at the beginning of some of the very large pours.

All cement meets the requirements of the Canadian Standards Assn. for standard portland cement and, according to A.S.T.M. Specifications, would be a Type I cement, but more coarsely ground. It is tested and accepted before being shipped. To do this, storage is reserved for the Commission at the cement mill and resident inspectors are there at all times sampling the daily run and checking shipments. Testing is done in the Commission's laboratories at Toronto and cement is accepted on a 7-day test.

Fine aggregate

The fine aggregate is a natural sand found near the job. It is inclined to be coarse and at times does not meet the specifications. When this occurs a fine sand is added at the mixer to give the required grading. This is possible as the mixing plant has a spare bin and scale through which it can be handled.

The use of blended sand may require some explanation. The Commission's experience with hydraulic structures built from the coarser sands are that they are neither as watertight nor as durable as those in which the sands are finer graded. Hence, its specifications require that the fine aggregate shall have a minimum of 12 percent passing the No. 50 sieve and where it can be obtained economically, a percentage between 15 and 20 is preferred. Since in many parts of Ontario the sands available for concrete are coarser than this, the practice has been adopted where necessary, of blending a fine and coarse sand to give the desired grading. This is usually done at the batching plant by pro-

viding it with two sand bins and weighing into every concrete batch the quantity of each sand required to give the desired combination.

Coarse aggregate

The coarse aggregate came from a large deposit alongside the road built to service the McConnell dam. It was crushed and screened into three sizes, ¼-¾ in., ¾-2 in. and 2-3 in. The pit contained relatively little sand, and this was of poor grading and was wasted.

The pit area was level and easy to work. After stripping to a depth of about four feet, three 16-cu yd Le Tourneau's were used to draw pit-run gravel to the crushing plant hopper. One bulldozer at the hopper kept it full at all times. All the aggregate passed through the crusher and the oversize above 3 in. was grizzled out and returned to the crusher by an underslung conveyor. This plant turned out aggregate in excess of 3000 tons per day, and, when necessary, ran 24 hours in two shifts.

To handle the gravel, an extensive conveyor system was built. A conveyor extended from the pit to the screening plant, another from the screening plant to storage and a third from storage to mixing plant, a total of 4200 ft. A fourth conveyor spilled the unwanted sand into Bell Lake where there was ample room for its disposal.

Aggregate storage and handling

A single storage yard with a capacity for 300,000 tons was provided for both fine and coarse aggregates. Twin single Bailey bridge trusses, erected by caterpillar crane, were used to span the piles and carry the overhead conveyor. All these spans were erected on the ground and lifted into place, using two cranes for the purpose. Upon completion of the storage trestle, which was supported on small concrete footings, the ground was leveled off and a 7-ft diameter heavy gage corrugated steel pipe was built for a recovery tunnel. On the top of the pipe at regular intervals are 24-in. ports through which, by shutters hand-operated from the inside, the desired rock sizes and sand are fed onto the supply conveyor leading to the mixing plant hopper. Telephone communications from the charging floor at the mixing plant to the two operators in the recovery tunnel was used to keep all bins supplied with their respective sizes of stone and sand.

Inverted "V" trestles, built of modified Bailey frames, were used to support a Bailey truss carrying a belt conveyor over the aggregate storage (Fig. 6). In this arrangement the free space between trestles at ground level was about 175 ft which gave ample room so that the storage piles did not encroach on the supporting trestles. In this capacity, the Bailey bridge panels gave good service and eliminated the usual trouble from the aggregate sliding and breaking the supports of the overhead distributing conveyor.

Fig. 6—Bailey bridging supporting a belt conveyor over aggregate storage. Reclaiming conveyor in foreground

From the crushing plant to the end of the screening system, there are 975 ft of Bailey bridge trestles, 2000 ft more for the storage yard, which, with the interconnecting conveyors, makes a total of 4300 ft from pit to mixer.

Mixing plant

The mixing plant for the main dam and power house is located at the Quebec (northerly) end of the dam about 150 ft back from the edge of the escarpment. It is the plant used during the late war for the construction of the Shipshaw development in eastern Quebec. It is equipped with four 2-cu yd Smith mixers set around a common receiving hopper into which they empty, which in turn discharges onto either one or both of two 30-in. conveyor belts leading out over Bailey bridging to the main dam and power house (Fig. 7).

The charging floor is equipped with Johnson batchers, one for cement, four for aggregates and a spare used on occasion for the blending sand. Two operators and an experienced maintenance man control the operation of these batchers. The operators work alternate hours, by way of relief, on the tedious and exacting job of keeping four mixers operating at one time. On the top floor of the plant, a large six-compartment bin receives three sizes of gravel and two of sand, by conveyor from the aggregate storage and cement from silos nearby. A wooden tank of 10,000-gal. (Imperial) capacity, housed for winter protection and steam heated when necessary, supplies water for the mixers. Ten to twelve men per shift are required to operate this mixing plant, inclusive of the two men in the recovery tunnel at the storage yard.

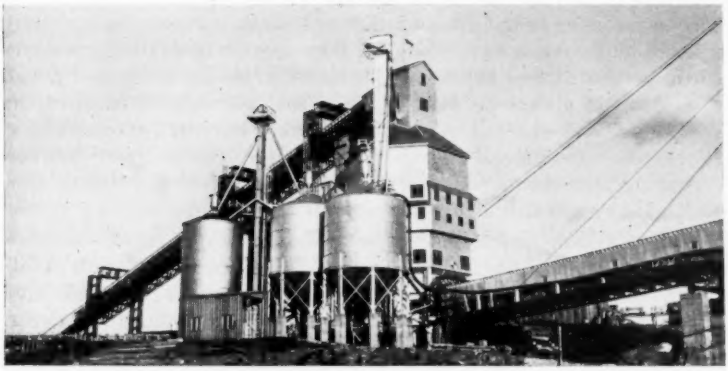

Fig. 7—Mixing plant and cement silos. Note again use of Bailey bridging for supporting conveyors.

A temporary mixing plant was built at the south end of the main dam while the main plant and overhead cableway were being erected, and was used to build about 600 ft of gravity section at that end of the dam; after the main plant was completed use of the temporary plant was discontinued. A third plant was built for concreting the McConnell dam. Both of these plants were of conventional construction and were equipped with two 2-cu yd Smith mixers and discharged through a common receiving hopper to 30-in. conveyor belts.

Distribution and placing

Primary distribution of the concrete is handled by 30-in. belt conveyors. Twin belts were used for the main dam to permit concreting simultaneously at two different points. The conveyors were carried on trestles built with Bailey bridging. At the main dam, the Bailey bridge was a separate structure, variously 100 to 135 ft high, built on concrete piers, on a center line 17 ft upstream from the face of the dam; at the McConnell dam, Bailey form cages served as piers to support the belt conveyor spans and housing. The former arrangement was the more convenient, allowing greater flexibility in building the forms and permitting better overhead access to them from the cableway.

The belt conveyors were equipped with movable trippers which discharged the concrete into steel chutes supported on light framework off the forms. These chutes fed into elephant trunks through which the concrete dropped vertically to the point of deposit.

Articulated drop chutes or elephant trunks are being used generally by the Commission in concreting the high lifts used in the construction of their dams, and with proper use, have been found very satisfactory.

They must never be hung at an incline or segregation occurs, particularly where 3-in. aggregates are used, but they may be swung a few degrees off the vertical at the discharge end to secure wider distribution. Enough chutes should always be hung in the forms to reach all parts of the area being concreted and minimize temporary interruptions in placing, so that the concrete can be kept "alive." In a dam of this size, when concreting near the foundation rock, this meant keeping active an area 40 ft wide by upwards of 100 ft long from front to back.

With the plant and methods described, as much as 46,000 cu yd of concrete per month have been placed at Des Joachims and daily yardages have exceeded 3000 cu yd. Sections of dam up to 50 ft high have regularly been placed in one continuous operation requiring, in a dam of this height and with vertical joints spaced at approximately 40 ft intervals, 8000 to 12,000 cu yd.

Treatment of joints

As soon as the surface has hardened sufficiently, horizontal joints between lifts are cleaned by air and water and the usual precautions are taken to get a bond between the old and new surfaces. In addition, steel plates, ⅛ in. thick by 12 in. wide and from 10 to 12 ft in length, are embedded to a depth of 6 in. about 8 ft back from the upstream face of the dam, and serve as a cut-off plane to stop leakage. Provision against sliding and shear, on all standard horizontal construction joints, is secured by stopping the lift in a series of steps, 6 to 8 ft wide and 6 in. high, rising from front to back of dam.

In vertical joints, a full ⅜ in. of emulsified asphalt is sprayed on the hardened concrete to take up expansion and act as a seal. This is done as soon as a bulkhead is stripped and long in advance of pouring the adjacent section of the dam. To further prevent leakage, a 6 x 6-in. vertical check leads from the top of the dam to an 8 x 12-in. gutter in the inspection tunnel floor. These gutters discharge below water on the downstream side or under a rock fill, if dry, to protect against freezing at the outlet.

Vertically below the gutter and on the rock foundation of the dam, a 12-in. half-round steel form or a 10 x 12-in. inverted plank box, with open side down, is carefully fitted to the rock. Pipe risers, 10 in. in diameter, lead upward from the top of this continuous box drain into the floor of the gutter to catch any leakage between rock surface and base of dam. Particular care must be taken in the concreting around the drain to see that it is not dislodged.

Concrete proportions and quality

In concrete used in hydraulic structures, durability is of more importance than strength and in the Commission's work it is the dominant

consideration in formulating the mix. However, no concrete is used in any exposed face which does not have a 28-day compressive strength of 3000 psi or better and engineering designs are based on this quality.

At Des Joachims, two classes of concrete, face and core, are used in the dam having average cement contents of 490 and 365 lb per cu yd respectively. The "face" is used for the front, top, back and base of the dam and the "core" for the rest. No sharp boundary exists between the two but a minimum of 6 ft of face concrete is maintained at all times backed by several feet of concrete in which both classes are variously intermingled. A third class of concrete with a cement content averaging 525 lb per cu yd and a smaller coarse aggregate, is used for the power house, headworks, penstocks, etc. The water-cement ratio (by weight) for core concrete ranges from 0.86 to 0.90, for the other two classes from 0.63 to 0.65.

Approximate proportions for the aggregates are determined by the Toronto laboratories from experimental studies conducted prior to construction but these may be varied in the field as job conditions make necessary. Actually, changes are few and within narrow limits. Considerable freedom is allowed the field in varying the proportions of the different sizes of coarse aggregate to take care of the varying quantities of each produced at the pit but these changes seldom affect the cement content of the concrete. Average compressive strengths of 6 x 12-in. cylinders cured under standard conditions are, for face concrete at 7 and 28 days respectively, 2980 and 4110 psi and for core concrete, 1740 and 2670 psi.

Winter concreting and protection

On practically every power development yet built by the Commission, a substantial volume of concrete has to be placed during the winter months and this has been so at Des Joachims. The methods there have followed recognized good practice—heated aggregates and water, housing of mixing and conveying equipment and protection of the concrete during its early life, but every effort is made to conserve BTU's. Aggregates are heated only sufficiently to keep them free from frost, the temperature of the concrete being regulated by the temperature of the mixing water. The concrete when placed is kept below 50 F if possible and the amount of protection given it is controlled by recording thermometers placed in the concrete itself just beneath the forms and inside the housings. For concrete which will later be covered by other concrete, only enough protection is given to prevent the surfaces from freezing, but wherever surfaces are to be exposed to water or weather a minimum curing period of 72 hours at 50 F is required and reinforced concrete is given whatever protection and heat is necessary for it to obtain the initial strength required.

CHENAUX DEVELOPMENT

At the Chenaux Development, the Ottawa River is divided into two channels by Limerick Island. The latter is again divided by a channel excavated during construction for diverting the river but forming a discharge channel in the completed development. Construction, therefore, involves the building of three dams and a power house, aggregating a total of about 225,000 cu yd of concrete of which 100,000 cu yd are in the power house.

The concreting methods follow much the same pattern as at Des Joachims except with these differences.

1. The coarse aggregate is a rock from the discharge channel crushed to two sizes, ¼ to ¾ in. and ¾ to 2 in.

2. Only one mixing plant was installed. It was equipped with two 2-cu yd Smith mixers discharging to a short inclined belt conveyor which could be used to supply either a concrete pump or trucks.

3. As none of the dams were over 60 ft high only two classes of concrete were used, one for the dam and the other for the power house, and concreting was continuous from rock to top.

4. The yardages of concrete in the dams were small as compared to Des Joachims and the structures were scattered so that an elaborate system of conveyor belts would have been costly. The contractor elected to place the concrete by twin pumpcrete installations which seemed to offer the most flexible and economical arrangement. The use of twin pumps was dictated by the fact that with two pumpcrete lines two classes of concrete could be placed simultaneously at widely scattered points with a minimum of preparatory work.

The use of concrete pumps with 2-in. crushed rock aggregate is the most interesting feature of the concreting operations at Chenaux. There was considerable skepticism at the start as to whether a pump would operate with the crushed rock available but there has been little trouble to date, although concrete at the main dam and power house has been pumped over 800 ft with a rise of 60 ft. For the Limerick Island bulkhead dam in the Quebec channel, concrete was trucked to a concrete pump set up close to the structure.

LA CAVE DEVELOPMENT

La Cave, the third and last of the Ottawa River projects to be carried out at this time, is now well under way. The work when completed will approximate a yardage of 370,000 of which 120,000 cu yd will be placed in the power house and headworks. It will also involve the relocation of some 37 miles of the Canadian Pacific Railway in the Province of Quebec.

The layout on this job is very similar to that at Des Joachims. The dam, which is 2600 ft long, is a straight line design except for a short gravity wing dam at each end. Primary placement of concrete will be by belt conveyor to short chutes from the tripper into elephant trunk hoppers for distribution in the forms.

This plant will have a head of 77 ft and the separate sections of the dam will be placed in two lifts. It may be mentioned here, although applicable to the other developments, that the initial pours to cover the rock are not all carried to the same height. They will vary from 5 to 6 ft alternately between them; in other words, the top of the initial pours (and the secondary 40-ft pours, if the dam is high) will have the appearance of a massive turret outline. This is done so that all horizontal construction joints are not in the same plane.

As at Des Joachims, gravel was found along the highway to the development, some 2800 ft from the mixing plant. The aggregate is crushed and screened into the usual three sizes, ½-¾ in., ¾-2 in. and 2-3 in. and conveyed to the storage yard. The mixing plant is equipped with a Johnson batching system and three 2-cu yd Smith mixers. For preliminary work, provision is now made to load trucks with concrete at the mixing plant for pumpcrete distribution to the eastern end of the dam and later for the power house itself.

CONCRETE CONTROL

Each of the three jobs described in this paper was preceded by a survey of available sources of aggregate in accordance with the Commission's practice on all jobs, small as well as large. Where commercially developed deposits exist, the survey is usually a simple matter of sampling, but where the jobs are in undeveloped areas as most of our power developments are, aggregates not only have to be found but the treatment, if any, required to process them to a usable form has to be worked out. This frequently requires air and ground reconnaissance, study of air maps for topography and flora and, after deposits are located, proof of the quality and quantity of the materials they contain. A small portable field laboratory is used for much of this work but final decisions are based on thorough testing in our Toronto laboratories where the necessary data is developed to enable procedures to be worked out and concrete mixes set.

Control laboratories are placed on all but the smallest jobs, (5000 cu yd or less and sometimes on these when they are in isolated localities), their equipment depending on the magnitude and duration of the project (Fig.8). Those on the larger jobs are capable of investigating most of the day-to-day problems arising in the course of the work and are so used even where the work duplicates, to some degree, investiga-

Fig. 8—Interior of field concrete laboratory equipped with 400,000 lb hydraulic compressive machine in background.

tions being conducted elsewhere. This practice has been found helpful, not only in getting wanted information, but in interesting the job personnel in better concrete and we find the experiments are followed closely by all concerned.

The system of inspection follows the usual pattern, inspectors on each shift at pit, quarry, mixing plant and at each form where concrete is being placed with other inspectors looking after curing and protection of the concrete from heat and cold, all supervised by a chief inspector responsible to the resident engineer. Inspection also includes the keeping of records of all concreting operations so referenced that we have a complete history of every part of every structure, including work stoppages, defects, surface cracking and other features that may have a bearing on the future serviceability of the concrete. Our inspection tries not only to see that the specifications are met but to provide a complete history of the concrete against future need.

DISCUSSION

As will be noted, many of the concreting methods of the Commission differ from those commonly used for structures of comparable size and type. The principal difference is the placing of the concrete in 50-ft lifts. This probably requires some explanation, for from it spring many of the other differences in construction methods in use on these jobs.

The use of high lifts originally came about from experience with hydraulic structures and retaining walls where it was early observed that much of the deterioration requiring repair and maintenance originated at construction joints, both vertical and horizontal. Of these, trouble at the vertical joints could be largely prevented by a combination of design and good workmanship in placing but in spite of all precautions, a tight horizontal joint could not be guaranteed, especially where the sections from front to back were shallow, as in retaining walls

and the smaller dams. From these observations it followed that the logical remedy was to eliminate the horizontal joint wherever possible which was first done as a planned procedure in the construction of the Chats Falls development in 1930, as reported by Trotter and Schnarr.* Here lifts up to 60 ft were successfully placed in one continuous operation but generally lifts have been limited to 50 ft.

In view of all that has been said and written about the advantages of 5-ft lifts, our practice of high lifts has naturally been viewed critically by concrete experts for in the eyes of many it is not only unorthodox but flaunts accepted concepts of sound practice. We have not been unmindful of this situation nor have we stubbornly pursued our way regardless, rather we have reviewed our procedures very critically. Early in the use of high lifts, we were faced with the seeming contradiction between our methods and those then coming into accepted use on like structures and we took steps to ascertain whether or not our structures showed any evidence of any dangerous condition such as longitudinal cracking. None was found in any structure examined. More recently, by means of the Soniscope described in a recent issue of the ACI JOURNAL by Leslie and Cheesman†, we have been able to more thoroughly survey our structures and the new method of examination has given the same result, *i.e.*, no evidence of any cracking or other condition that would involve their safety or usefulness.

The use of high lifts necessitates, on occasion, the placing of large volumes of concrete in one continuous operation, which naturally takes considerable time. In one dam recently completed, the maximum height of which was a little over 200 ft, several sections were as much as 40 ft wide, 140 ft front to back as well as 50 ft high and contained approximately 15,000 cu yd of concrete. The question naturally arises, "Can the working surface of the concrete be kept fresh over such large areas?" With careful preparation and proper organization, no great difficulty has been found either in placing this amount of concrete in one operation or in keeping the concrete surfaces in proper condition for receiving further concrete. Obviously, in so large an area some time must elapse between successive deposition of concrete in any one spot in the forms but usually this can be done within an hour even with the large form mentioned and no harm seems to result if this period is as much as 90 minutes.

High lifts bring up another question, that of temperature rise in these large masses. It is recognized that with the methods we follow the temperatures existing in the center of our structures are close to the

*Trotter, H. L. and Schnarr, Wilfrid, "Concreting Problems—Chats Falls Power Development," ACI JOURNAL, Feb. 1933, *Proc.* V. 29, pp. 249-274.

†Leslie, J. R. and Cheesman, W. J., "An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures," ACI JOURNAL, Sept. 1949, *Proc.* V. 46, pp. 17-36.

theoretical maximum and that these temperatures remain high for months and years, but our investigations, as already stated, have not revealed more than superficial surface cracking, except in one or two instances and none of a type endangering the structure. We may be wrong in not limiting these temperatures but results seem to indicate otherwise, for in the almost 20 years we have been following this practice no adverse effects have been found although some of our structures have exceeded 200 ft in height and all have used Type I cement.

The use of high lifts obviously requires the use of high forms with internal ties; this precludes the placement of concrete by cableway and buckets and necessitates distribution of concrete from mixer to form by some other means. In the different jobs described various methods have been used, belt conveyors, concrete pumps, elephant trunks, trucks and even sloping chutes. It is our experience that good concrete will be the end product of any of these methods—even with the much maligned chute—if used intelligently, and under proper supervision. Each method, including the bottom dump bucket, has its advantages and its limitations, and our practice is to use any or all of them as circumstances indicate, always subject to the close supervision of men who know how to use them properly.

Title No. 46-38

Important changes in the ACI Building Code are suggested in

Proposed Design Specifications for Two-Way Floor Slabs*

By N. M. NEWMARK† and C. P. SIESS‡

SYNOPSIS

A new design specification for two-way concrete floor slabs is presented. It is proposed as a replacement for the methods currently contained in Section 709 of the ACI Building Code (ACI 318-47). This new specification is based on analyses of continuous rectangular slabs carrying a uniformly distributed load. Account is taken of the continuity of the slab, of the torsional stiffness of the beams, and of the deflection of the beams.

INTRODUCTION

The new specification follows in general the form of Method 2 in Section 709 of the ACI Code, but is patterned to some extent on the provisions of Chapter 10, "Flat Slabs." However, it differs from previous procedures, both in principle and in detail, in several respects.

For slabs supported on reinforced concrete beams built monolithically with the slab, or by steel beams fully encased in concrete, the specifications for design are derived from those proposed in the paper "Rational Analysis and Design of Two-Way Concrete Slabs" by the authors, published in the December 1948 ACI JOURNAL. For this case the effects of torsional stiffness of the beams are considered and lead to moment coefficients for the slab which are the same for all panels, whether interior, edge, or corner.

For slabs supported on bare steel beams, not integral with the slab and thus incapable of providing torsional restraints, a different set of moment coefficients are obtained. These coefficients are greater than those for the case of concrete beams, and increase progressively as the number of discontinuous edges possessed by a panel is increased. Specifically,

*Presented at the ACI 46th annual convention, Chicago, Ill., Feb. 21, 1950. Title No. 46-38 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 8, Apr. 1950, *Proceedings*, V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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separate sets of moment coefficients are proposed for the following four cases: (1) an interior panel, with all edges continuous; (2) an edge panel with a short edge discontinuous; (3) an edge panel with a long edge discontinuous; and (4) a corner panel, with two adjacent edges discontinuous.

MOMENT COEFFICIENTS

In principle the proposed new specifications differ from those in the present ACI Code in that moment coefficients are given for average moments on sections extending the full width of a panel. Furthermore, the magnitudes of the moment coefficients specified are somewhat less than those currently used, as the result of considering the action of the structure at loads approaching failure and the redistribution of moments which can take place at that stage. These considerations have long been the basis for the common design procedures for flat slab floors. For such floors it was shown* years ago that the "total" moment which governs the design, can be computed by statics, and is in fact the total statical moment for uniform load on a panel. It follows, therefore, that the moment M_o used in Chapter 10 of the ACI Code is only 72 percent of the total statical moment, and no consideration is given in the moment coefficients in that chapter to the possibility of maximum moment on different sections arising from different cases of loading. *The total moment provided for is only 72 percent of the least possible moment* which can exist for the given intensity of loading.

However, in contrast to this point of view, it has been required that two-way slab floors be designed for more than 100 percent of the total statical moment, the excess over 100 percent arising from the consideration of different loading conditions for positive and negative moments.

The writers feel that such conservatism is unwarranted and unjustified by experience or test. However, they do not feel that it is desirable to approach for two-way slabs the lack of conservatism even responsible engineers are willing to accept for flat slabs. The proposed specification is intended to strike a reasonable balance.

There are two essential differences in the assumptions underlying the design of these two types of structures: (a) the flat slab is designed for a uniform load on all panels, while the two-way slab has been designed for combinations of loaded panels producing maximum moments; (b) an arbitrary 28 percent reduction of the total moment in flat slabs has been made. In the development of a design procedure for the two-way slabs, the authors chose to consider only (a) above.

For uniform loads in all panels, the sum of the positive and negative moments in the slab and in the beams is $0.1250\ wab^2$, or $0.1250\ wba^2$,

*Nichols, J. R., "Statical Limitations Upon the Steel Requirements in Reinforced Concrete Flat Slab Floors," *Trans.* A.S.C.E., V. 77, 1914, pp. 1670-81.

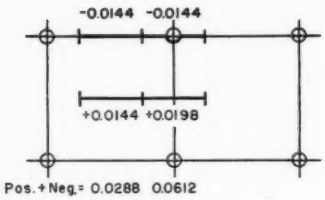
depending on the span considered. For partial loadings, the sum of the positive and negative moments is greater than this value by various amounts depending on the type of loading considered. This design procedure is based on the decision that the total moment carrying capacity of the slab and the beams should be sufficient to resist the moments resulting from uniform load over all panels, taking into consideration the possible redistributions that may occur at high loads. However, it also recognizes the probability of occurrence of partial loadings and takes into account the overstresses which might exist. This has been done by apportioning the total moment in the slab and in the beams to the positive and negative moment sections on the basis of the maximum positive and negative moments produced by partial loadings. For slab moments, the type of partial loading considered was what the authors chose to call "single panel" loading, rather than the somewhat more unfavorable but less probable "checker-board" loading. For the beams, the desired apportionment has been obtained by requiring that the beams be designed for the moments specified for use with beams and one-way slabs. These moments are based on partial loadings.

In selecting the moment coefficients to be used for the design of the slab and the beams the following considerations were applied: (a) The slab is the stronger part of the combination and therefore should be made to carry the greater share of the load. (b) To reduce the slab moments and then to make the beams carry the remainder of the total static moment, leads to a result inconsistent with (a). (c) However, when the major carrying elements are the beams, as in the case of the long direction of rectangular panels, the design should be based on considerations similar to those governing a structure made up wholly of beams.

These considerations led to the provisions for beam moments in section (*g*) of the proposed specifications. The factor of 80 percent for the beams has the effect of making the total static moment provided for in slab and beams about equal to the total static moment due to the load, but the moments are carried more by the slab and less by the beams because the slab has inherently a greater margin of safety. However, the factor is increased to unity for the beams when the moment is practically all carried by the beams.

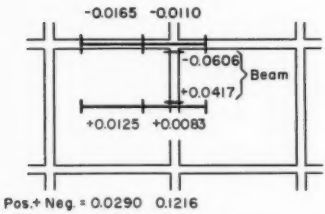
The relative amounts of moment carried by the different elements of the structure are portrayed in Fig. 1, 2 and 3. In these figures, (a) shows the moments on various sections, column and middle strips, in a flat slab, according to the ACI Code; (b) shows the corresponding moments, including those in the beams, for a two-way slab designed in accordance with the present Method 2 of Section 709; and (c) shows the moments for the proposed design specifications. In comparing these figures, one

Note: All moments in terms of wb^3; b/a = 1.0



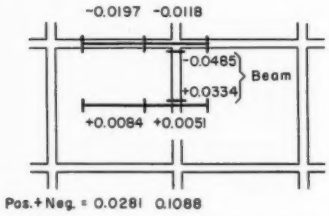
Total Slab Moment = 0.0900
Total Beam Moment = 0
Total Moment = 0.0900

(a) Flat Slab



Total Slab Moment = 0.0483
Total Beam Moment = 0.1023
Total Moment = 0.1506

(b) Two-Way Slab – Present Code Provisions (Method 2)



Total Slab Moment = 0.0450
Total Beam Moment = 0.0819
Total Moment = 0.1269

(c) Two-Way Slab – Proposed Code Revision

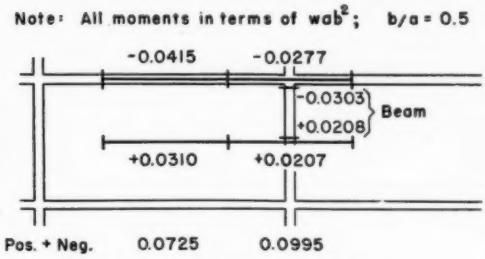
Fig. 1—Corresponding moments in square two-way slabs and flat slabs

should note that the slab column strip together with the beams performs the same function in the two-way slab that the slab column strip alone does in the flat slab.

SPECIAL FEATURES

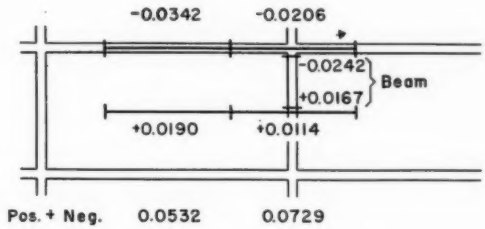
There are two features of the proposed specification which may require further explanation. The first concerns the provision for alternative methods of design. This parallels the present provisions for flat slab construction, and permits the use of accepted methods of analysis to cover cases not specifically treated in the code.

Fig. 2—Corresponding moments in short span of rectangular two-way slabs as computed by present code and proposed revision



Total Slab Moment = 0.1209
Total Beam Moment = 0.0511
Total Moment = 0.1720

(b) Present Code Provisions (Method 2)



Total Slab Moment = 0.0852
Total Beam Moment = 0.0409
Total Moment = 0.1261

(c) Proposed Code Revision

The second provision concerns the limitation on depth of the slab. In the proposed specification, the minimum slab thickness in terms of the span length is stated as 1/40 of the length of the shortest span. This provision may be somewhat over simplified, especially in that it does not take into account the degree of restraints at the ends of the span. A somewhat more complicated but more rational provision can be stated in the following form:

(a) For Case A, in which the beams have torsional stiffness:
Minimum thickness = 1/45 x shorter span.

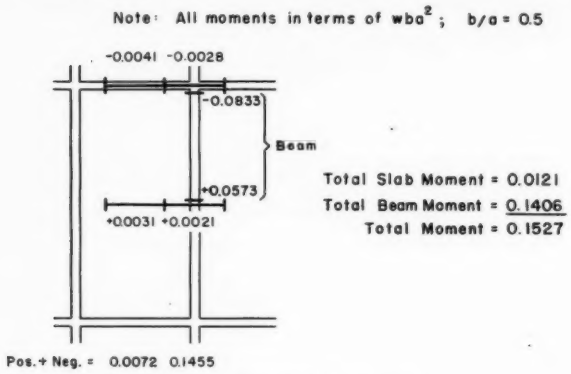


Fig. 3—Corresponding moments in long span of rectangular two-way slabs as computed by present code and proposed revision

(b) For Case B, in which the beams have no torsional stiffness:

Minimum thickness = 1/40 x shorter continuous span.

Minimum thickness = 1/35 x shorter discontinuous span.

The numerical values for the coefficients in the above example may need revision to bring them more into line with the present specifications. This is not necessarily desirable, however, because the present specifications give entirely too much weight to the shape of the panel in setting the minimum thickness. The value specified in section (*i*) represents in the authors' opinion a reasonable compromise.

CONCLUSION

In conclusion, it is intended to present a rational and only reasonably conservative specification for the design of two-way slabs for building floors. When both the slab and the beams are considered, the proposed specification permits a considerable economy over the present code provisions. This increase in economy does not require a loss in safety; the new requirements are still much more conservative than those governing the design of flat slab floors.

PROPOSED SPECIFICATIONS FOR TWO-WAY FLOOR SLABS

709—Two-way slabs with supports on four sides

(*a*) *General*—This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with fillers of hollow concrete units or clay tile, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. For design of the slab, two types of construction are considered, as follows:

Case A

Slabs supported on reinforced concrete beams built monolithically with the slab, or steel beams encased in concrete placed monolithically with the slab, in which case the beams provide restraint against rotations of the edges of the slab.

Case B

Slabs supported on steel beams, not encased in concrete placed monolithically with the slab, in which case the beams provide no restraint against the rotations of the edges of the slab.

(*b*) *Scope*—This section provides for two methods of design of two-way slab structures.

(1) Any type of two-way slab construction may be designed by application of the principles of continuity, using recognized methods of structural analyses.* In such analysis, the numerical sum of the positive moments and the average of the negative moments, for both slab and beams in any span, shall not be less than the total static moment in that span. However, the provisions of Sections 709 (*d*), (*h*), and (*i*) apply in every case.

(2) Where the construction falls within the limitations outlined in the next paragraph, Section 709 (*c*), the method of design given in Sections 709 (*d*) through (*i*) may be used.

*Acceptable methods of analysis are given in the following papers: Bertin, R. L., di Stasio, Joseph and Van Buren, M. P., "Slabs Supported on Four Sides," ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 537-555 and Siess, C. P. and Newmark, N. M., "Rational Analysis and Design of Two-Way Concrete Slabs," ACI JOURNAL, Dec. 1948, *Proc.* V. 45, Appendix, pp. 304-315.

(*c*) *Limitations*—(1) The slab is continuous for at least two panels in each direction.

(2) Span lengths in the same direction in adjacent panels differ by not more than one-half of the shorter span.

(3) The live load is assumed to be distributed uniformly over the entire area of each loaded panel.

(*d*) *Design sections*—The critical sections for negative moment shall be taken along the edges of the panel at the faces of the supporting beams for Case A, and at the ends of the effective span for Case B.

The critical sections for positive moment shall be taken along the center lines of the panels.

For placement of reinforcement, the width of the slab at each critical section shall be divided into a middle strip, covering the central half of the width, and two column strips, covering the outer quarters. Where the ratio of short span to long span is less than 0.5, the middle strip in the short direction shall be considered as having a width equal to the difference between the long and short span, the remaining area representing the two column strips.

For bending up truss or bent bars used for positive moment reinforcement and for the calculation of anchorage requirements for straight bars similarly used, the lines of contraflexure shall be assumed to be at a distance of 1/6 the shorter span from each edge of the panel.

For computing anchorage requirements for negative moment reinforcement, the lines of contraflexure shall be assumed to be at a distance of 1/5 the shorter span from each edge of the panel.

For Case A, effective span lengths of panels may be taken as the clear span between faces of supporting beams. For Case B, effective span lengths shall be taken as the center-to-center distance between supports or as the clear span plus twice the thickness of the slab, whichever value is smaller.

(*e*) *Bending moment coefficients for slab*—Coefficients for average bending moments in the slab are given in Tables 1A and 1B. These coefficients when multiplied by wb^2 give the average bending moment per unit of width of slab, where w is the load per unit of area, and b is the effective length of the short span. Coefficients are given in both tables for panels having various values of b/a, the ratio of the short span to the long span. Interpolation between the values of b/a is permissible.

Case A

The coefficients in Table 1A apply to slabs satisfying the conditions for Case A of Section 709 (*a*), and are the same for all panels, whether

TABLE 1A—MOMENT COEFFICIENTS FOR RECTANGULAR PANELS SUPPORTED ON FOUR SIDES AND BUILT MONOLITHICALLY WITH THE SUPPORTS (CASE A)*

Moment	Span	Ratio of span lengths, b/a						
		0	0.5	0.6	0.7	0.8	0.9	1.0
Negativemoment over interior beams	Short	.0780	.0548	.0501	.0455	.0408	.0362	.0315
	Long	.0390	.0390	.0387	.0378	.0363	.0342	.0315
Positive moment	Short	.0472	.0304	.0270	.0236	.0202	.0169	.0135
	Long	.0135	.0135	.0135	.0135	.0135	.0135	.0135

Coefficients are for average moments across entire width of panel. These coefficients, when multiplied by wb^2, give the average moment per foot of width. w = load per unit of area, b = short span, as defined in Sec. 709 (*d*), and a = long span, as defined in Sec. 709 (*d*).

*In any span an adjustment in positive and negative moment provided for in the design of the slab may be made as follows: The average positive moment may be increased by any desired amount and the average negative moment decreased not more than an equal amount but not more than 0.0030 wb^2.

TABLE 1B—MOMENT COEFFICIENTS FOR RECTANGULAR PANELS SUPPORTED ON FOUR SIDES AND NOT BUILT MONOLITHICALLY WITH THE SUPPORTS (CASE B)*

Location of panel	Moment	Span	Ratio of span lengths, b/a						
			0	0.5	0.6	0.7	0.8	0.9	1.0
Interior— All edges continuous	Negative moment over interior beams	Short	.0815	.0570	.0521	.0472	.0423	.0374	.0325
		Long	.0435	.0435	.0431	.0417	.0395	.0365	.0325
	Positive moment	Short	.0535	.0325	.0289	.0253	.0217	.0181	.0145
		Long	.0145	.0145	.0145	.0145	.0145	.0145	.0145
Edge— One short edge discontinuous	Negative moment over interior beams	Short	.0815	.0600	.0557	.0514	.0471	.0428	.0385
		Long	.0435	.0435	.0431	.0421	.0403	.0377	.0345
	Positive moment	Short	.0535	.0360	.0320	.0280	.0240	.0200	.0160
		Long	.0150	.0150	.0150	.0150	.0150	.0150	.0150
Edge— One long edge discontinuous	Negative moment over interior beams	Short	.0915	.0630	.0573	.0516	.0459	.0402	.0345
		Long	.0550	.0550	.0536	.0505	.0465	.0425	.0385
	Positive moment	Short	.0730	.0370	.0326	.0282	.0238	.0194	.0150
		Long	.0160	.0160	.0160	.0160	.0160	.0160	.0160
Corner— Two adjacent edges discontinuous	Negative moment over interior beams	Short	.0915	.0660	.0609	.0558	.0507	.0456	.0405
		Long	.0570	.0570	.0565	.0543	.0499	.0453	.0405
	Positive moment	Short	.0730	.0420	.0370	.0320	.0270	.0220	.0170
		Long	.0170	.0170	.0170	.0170	.0170	.0170	.0170

Coefficients are for average moments across entire width of panel.

*In any span an adjustment in positive and negative moment provided for in the design of the slab may be made as follows: The average positive moment may be increased by any desired amount and the average negative moment decreased not more than an equal amount but not more than 0.0030 wb^2.

located in the interior, at an edge, or at a corner. Negative moment at an exterior or discontinuous edge, along which the slab is supported by a beam meeting the requirements of Case A, shall be taken equal to two-thirds the value given in Table 1A for negative moment over an interior beam.

Case B

The coefficients in Table 1B apply to slabs satisfying the conditions for Case B of Section 709 (*a*). Separate coefficients are given for interior, edge, and corner panels. The negative moment at an exterior edge, if no restraint is provided at that edge, may be taken equal to zero. If the supports at an exterior or discontinuous edge meet the requirements for Case A, the negative moment at that edge shall be taken equal to two-thirds of the value given in Table 1A for negative moment over an interior beam; and the moments at other sections shall be taken equal to those given in Table 1B for an interior panel.

The average moment per foot of width in the middle strip shall be 1.25 times the moment determined from the coefficients in the tables. The average moment in the column strips shall be 0.75 times the value from the tables. Any reasonable variation of moment across the middle and column strips may be assumed, subject to the following limitations: (1) The moment is least at the edge of a panel and increases toward the center. (2) The moment at the edge of a panel is not less than one-half the average moment from Table 1A or 1B. (3) The average moment across each strip is not less than that determined from the table. A uniform distribution of moment in each strip will meet these requirements.

Corner reinforcement

For Case B, where the slab is not securely attached to the supporting beams or walls, special reinforcement shall be provided at exterior corners in both the bottom and top of the slab for a distance in each direction from the corner equal to 1/5 the shorter span. The effective amount of such reinforcement per foot of width in both the bottom and top faces of the slab shall be equal to that required for positive moment in the middle strip. The effective amount is equal to the normal area of the steel multiplied by the sine of the angle which the bar makes with critical section. In the top of the slab the critical section is perpendicular to the diagonal line bisecting the angle at the corner; in the bottom of the slab it is parallel to this diagonal line.

(*f*) *Distribution of unequal negative moments at supports*—In applying the moment coefficients of Table 1 to adjacent panels of varying dimensions or unequal loading, the negative moments on either side of a supporting beam may differ materially. When this condition exists, two-

thirds of the unbalanced negative moment in Case A and all of the unbalanced negative moment in Case B shall be distributed to the two spans in proportion to their respective stiffness. If the span lengths do not differ by more than 50 percent of the shorter span, it may be assumed that the stiffnesses are proportional to the value of t^3 where t is the thickness of the slab.

No adjustment need be made in the positive moments.

(*g*) *Bending moments and shears in supporting beams*—The loads on the supporting beams for a two-way rectangular panel may be assumed as the uniformly distributed load within the tributary areas of the panel bounded by the intersection of 45-degree lines from the corners with the median line of the panel parallel to the long side.

This distribution may be transformed to equivalent uniform loads per unit length of beam, due to one loaded panel only, giving the same maximum moment in a simply-supported beam, as follows:

$$\text{For the short span, } \frac{wb}{3}$$

$$\text{For the long span, } \frac{wb}{3} \times \frac{3 - (b/a)^2}{2}$$

Bending moments and shears in the beams in each direction shall be computed according to the requirements of Sections 701 and 702, for a uniform load equal to Q times the equivalent uniform loads given above. The factor Q may be taken as 0.8 for the short span or for both spans of a square panel, and shall be increased linearly for the long span from 0.8 to 1.0 as b/a decreases from 1.0 to 0.5.

(*h*) *Shear in slab*—Shearing stresses in the slab shall be computed on the assumption that the load is distributed to the supporting beams in accordance with paragraph (*g*) above.

(*i*) *Minimum thickness and reinforcement of slab*—The slab thickness in any panel shall not be less than 4 in., nor less than 1/40 the length of the shorter span of the panel. The spacing of reinforcement shall not be more than three times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

Discussion of a paper by N. M. Newmark and C. P. Siess:

Proposed Design Specifications for Two-Way Floor Slabs*

By PHIL M. FERGUSON, WALTER HESS, L. J. MENSCH, R. C. SANDBERG AND AUTHORS

By PHIL M. FERGUSON†

Discussion of this paper necessarily involves the earlier paper by the same authors on the analysis of two-way slabs.‡ The major portion of the proposed specification relates to slabs and seems to follow logically from this analysis. The analysis method appears to be a distinct forward step, but it must be kept in mind that the theoretical basis for the distribution procedure that is applied to these slabs is not yet a matter of public information.§ The strong and weak points in the basic theory and the accuracy obtained by using average edge moments and average angles of rotation cannot be evaluated until the basic theory is published. In the meantime, the serious consideration generally being given to the method is simply evidence of the high esteem in which the authors are held. Serious consideration of this paper as a specification should await the publication and public appraisal of this basic theory.

The proposed specification for the supporting beams is not acceptable in the writer's opinion. It suggests a method for the design of these beams which puts them into a design class different from all other beams. Although it is not so designated, the method proposed is based on limit design ideas.

The specification proposes that the calculated beam moments be reduced by a factor Q, which for square panels is 0.8. It is indicated that about 5 to 7 percent of this 20 percent reduction represents moment transferred to the slab by the ordinary deflection of the beams. If this is so, specified equivalent uniform loads for the short span should be restated at approximately 95 percent of those proposed. The other 15 to 13 percent of the proposed 20 percent reduction results from consideration of the possible redistributions that may occur at high loads. Some of this redistribution would tend to be to the slab, but to a slab already overstressed. Hence it seems that a con-

*ACI JOURNAL, Apr. 1950, *Proc.* V. 46, p. 597. Disc. 46-38 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

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‡"Rational Analysis and Design of Two-Way Concrete Slabs," ACI JOURNAL, Dec. 1948, *Proc.* V. 45, p. 272.

§Siess, C. P. and Newmark, N. M., "Moments in Two-Way Concrete Floor Slabs," *Bulletin No. 385*, University of Illinois Engineering Experiment Station, was not announced until shortly after the closing date for discussion of this paper.

siderable part of the redistribution would have to be redistribution within the beam itself.

The writer can see little justification for designing these beams for redistribution and designing other beams for no redistribution. This question of policy should be handled as a unit for *all* beams and in a consistent manner.

The writer would not like to be classified as totally opposed to the idea of limit design as reflected in reduced design moments or increased unit stresses. He realizes that the factor of safety of continuous beams is increased by redistributions that take place at high loads. As the redistribution process becomes generally better known, he believes that some recognition of this increased strength will be accepted.

Nevertheless, the increase in the factor of safety thus accomplished by redistribution is generally a low grade or "second-class" increment. It is accompanied by increasing deflections and a permanent set in the beam. In a way it is akin to the idea of basing the factor of safety for a simple tension steel hanger upon the ultimate strength of the steel instead of upon the yield point strength. Most engineers would value each 1000 psi of strength above the yield point much less than a smaller increment within the yield point. The two zones are not directly comparable. In approaching the question of limit design of beams, when the time does come that ACI is ready to consider it for *all* beams, might it not be well to break the final apparent factor of safety into two parts: one, a first-class part in the zone of elastic action as we think of it; the other, a second-class or inferior part in the zone of redistribution and highly inelastic action. Admittedly, in reinforced concrete one has to use the term elastic in an approximate manner, but the distinction between the two zones is real; the inelastic action is of a different order of magnitude. This inelastic second part of the factor of safety could then be discounted or weighed at less than its nominal value for a proper evaluation of the total factor of safety.

In the matter of the beam shear calculations, the proposed specification seems unduly indirect. In a situation where simple statics is entirely adequate, the writer sees no reason to use an equivalent load that is too large (because it is based upon the needs for moment) and then to develop a factor to correct the error. The simple beam shear based directly on the loading of the first paragraph of Art. 709(*g*) seems much more reasonable. It would also give a better picture of the variation in shear along the beam length. Designers who wish can add an estimated 5 to 10 percent allowance for the effect of continuity.

This discussion has been critical of certain limited sections of the proposed specification. Nevertheless, the writer has the feeling that the final evaluation of this paper, after publication of background materials, will mark it as a real contribution to reinforced concrete progress.

By WALTER HESS*

The purpose of this discussion is not to question the theory leading up to the proposed specifications, but to make some comparisons and offer suggestions as to the simplification of the proposed code. The authors have done a masterful job of computation work, as well as presenting the whole subject in an understandable form.

The writer believes that the comparison of actual moments of some particular problem would be more enlightening than a comparison of moment factors, particularly when the factor is a function of the second or third power of a span or spans. Thus, the writer has selected a panel out of the authors' previous article† on two-way slabs, a 10 x 20-ft slab, designated as panel No. 3, and applied a 400 psf total load, live load being three times the dead load. Positive and negative moments were calculated on the basis of Methods No. 1 and 2 of the ACI Code, also according to the authors' article in the Dec. 1948 ACI JOURNAL, and also on the basis of the proposed specifications.

The results are presented in Table A, all values being in ft-kips over the entire width of the panel.

TABLE A—COMPARISON OF MOMENT

Method	Neg. M_a	Neg. M_b	Pos. M_a	Pos. M_b
Method No. 1	64.73	21.58	37.13	12.38
Method No. 2	55.33	11.00	41.33	8.33
Moment distribution	44.48	12.60	32.10	4.62
Proposed code	43.84	15.60	24.32	5.40

The following notes should serve to clear up some of the values indicated in the table.

1. *Method No. 1*—Moment coefficients were used from Table 1, modified by 1/11 for negative moment and 1/16 for positive moment, the positive moment being further modified by the requirement in Section 709 (*c*), Item 2. Negative M_b was assumed as one-third of M_a from the requirement in Item 1 of the same section.

2. *Method No. 2*—Moment coefficients were taken from Table 3, modified by the requirement in Section 709 (*b*) under Method No. 2.

3. *Moment distribution*—This was based on the Newmark and Siess article in the Dec. 1948 ACI JOURNAL. All factors were based on a typical interior panel, using only one carry-over factor, that of C_{aa}. In determining live load moments, only the panel in question was loaded, and surrounding panels were assumed as being the same size.

4. *Proposed specification, Case A*—All moment factors were taken from Table 1A of the proposed specifications.

The proposed and present codes, as well as previous codes are rather confusing as to nomenclature, particularly as to span designation. Further, the average designer comes across many special applications to this problem, such as concentrated loads, adjacent one way slabs, etc. No code is economically convenient if the designer has to make a study of it every time he comes across a problem applying to that code.

*Sinclair Refining Co., East Chicago, Ind.

†"Rational Analysis and Design of Two-Way Concrete Slabs"' ACI JOURNAL, Dec. 1948, *Proc.* V. 45, p. 272.

Therefore, the writer suggests that the authors revamp their proposed code, setting up a table of direct load distributions based on the ratio of span lengths, modifying these factors with other factors based on slab stiffness, redistribution, etc. Distribution of reinforcing steel in the slab and beam strips might be included, although the writer does not believe this to be economical in a construction sense. From here, the designer could work to his own discretion as to moment distribution, placement of live loads, etc. Some limitation should be specified as to what span ratio the two-way slab economically becomes a one-way slab.

In general, all present and proposed codes are more or less in agreement as to beam loads surrounding a two-way slab, but the writer believes that a table of factors converting triangular and trapezoidal loading to uniform loading would be in order.

By L. J. MENSCH*

The writer does not agree at all with anything presented by the authors. In the 1911 ACI *Proceedings*† the writer indicated what the moments are in a flat slab construction when it is supported by columns.

Now, reference is made in the paper to a flat slab supported on pin points. It is not supported on pin points. Each slab is a rigid frame between the two columns and the flat slab. The writer showed 39 years ago that the columns used then made the slab nearly fixed over the columns. So you still have—even if only one panel is loaded—a fixed condition at the columns.

Of course, flat slabs may be built with pipe column supports. Then, of course, you do not have a fixed condition over the columns because a pipe column has comparatively small moment of inertia; but when you use a concrete column that is $L/15$ in diameter of the span, you have a fixed condition. All the tests the writer has seen showed that the columns cracked before the slab cracked; so it shows that the columns must take up a great moment.

In regard to so-called two-way slabs, the authors didn't consider the tests and the literature on the subject in the generation before this. The thought seems to be that to be up to date, one has to know what has been written and done in the last 20 years, but this subject was investigated both theoretically and by tests 50 years ago, and we didn't hear a thing about the great masters who devoted many years of study to it.

By R. C. SANDBERG‡

The writer is a little confused when a comparison is being made between a concrete flat slab, as we usually think of it, and a two-way slab which might be supported by a perimeter of walls—not beams, but walls.

The writer would assume that the positive moment for flexure along this wall would be zero, but according to the paper we would get some moment

*Evanston, Ill.

†"The Calculation of Reinforced Concrete Flat Plates Supported by Four Columns," ACI *Proceedings*, V. 7, 1911, p. 205.

‡Architect, Rock Island, Ill.

at this point if it were a beam or flat slab. Certainly if it is a concrete flat slab, we would have our greatest moment across a transverse section across column band or location of wall as far as the positive moment is concerned. We had a peculiar experience in one instance right at this point in the slab where the positive moment should be the greatest. We had four columns at the four corners of a panel; it was a type of slab similar to a flat slab except that it was of a uniform depth. There were tile fillers in the slab so that in two directions there were ribs about 4 in. wide. The people who were putting in the refrigeration system had forgotten a conduit so they cut out all of the top slab at the point of maximum positive moment, across the whole width of column strip or band. The writer was notified about the cutting of the slab and hurried out to the job expecting to find some dire results. Not only was the slab cut across the column strip for the full depth of compressive section, but all the machinery had been moved out over the point where the slab had been cut out.

It didn't fail; it didn't seem to show any signs of bad deflection. It was evident that compressive stresses in bending were resisted by the concrete top slab of the mid-strip. The writer concluded then that perhaps slabs should be designed so that they would have greater provision for positive moment in this section of the mid-strip. We have a ratio, mid-strip moment and column-strip moment distribution of about 3 to 4 according to present codes. Since the incident described, the writer has designed these slabs so that there is an equal ratio of the steel in the section across mid- and column-strips.

Why do we, when the slab is resting on walls, make as high a provision for positive moment across the column strip? How about the condition where the concrete compressive area was removed? Shouldn't we provide for greater positive moment at the mid-strip; certainly the compression work done by this cut-away slab must be transferred to the adjoining panel and in the mid-strip.

AUTHORS' CLOSURE

Mensch is concerned about the fact that the flat slab in the illustrative example of Fig. 1a is assumed to be supported on "pin points" rather than on columns having a capital of diameter c as is the usual case. This point may have troubled some readers unnecessarily. However, it is noted that the comparisons of Fig. 1 would be unaffected by the diameter of the support. The total static moment for a flat slab was expressed approximately by J. R. Nichols as

$$M_o = 0.1250\, wb^3 \left(1 - \frac{2}{3}\,\frac{c}{b}\right)^2$$

where c is the diameter of the column capital. The corresponding expression in Sec. 1003 of the ACI Code is

$$M_o = 0.09\, wb^3 \left(1 - \frac{2}{3}\,\frac{c}{b}\right)^2$$

It can easily be seen that the ratio of these two expressions is 0.72, no matter what the value of c is. In Fig. 1a, the value of c was taken equal to zero so that the coefficient for the total static moment would be 0.1250 for all three cases considered.

If the slab supported on perimeter walls, as mentioned by Sandberg, is a single panel, it would fall outside the limitations of Sec. 709(*c*) of the proposed specification and would have to be treated by other applicable procedures. Furthermore, if a continuous slab is supported on walls, the proposed coefficients in Table 1B would also be in error since they are intended to apply to a slab supported on flexible beams. In this case, however, the moment distribution procedure described in a previous paper* by the writers would be applicable. Likewise, it does not seem reasonable that any simple design procedure can be expected to provide for the possibility that holes may be cut in the slab at any location. Sandberg's modification of the flat-slab positive moment coefficients in middle and column strips appears to be based on the assumption or prediction that any holes that may be cut in the slab will be located in the column strip.

Hess finds comparisons in terms of total moments for a specific example to be helpful. Although the writers prefer comparisons of coefficients as illustrated in Fig. 1-3, others may prefer the tabulation made by Hess. However, to obtain the entire picture, such comparisons should be made for several ratios of spans and should include comparisons with the total static moments. For this purpose the moments in the beams must be considered. It may be helpful to add the beam moments to the figures obtained by Hess from the specifications proposed in the paper. For the short span the moments are:

Slab, positive	=	24.3 ft-kips
Slab, negative	=	43.8 ft-kips
Total, slab	=	68.1 ft-kips
Beams, positive	=	13.3 ft-kips
Beams, negative	=	19.4 ft-kips
Total, beams	=	32.7 ft-kips
Total	=	100.8 ft-kips

In this case, two-thirds of the moment resistance is provided in the slab, and the sum of all moments is about one percent greater than the static moment of 100.0 ft-kips. This is in close agreement with the data in Fig. 2c.

For the long span the moments are:

Slab, positive	=	5.4 ft-kips
Slab, negative	=	15.6 ft-kips
Total, slab	=	21.0 ft-kips
Beams, positive	=	91.4 ft-kips
Beams, negative	=	133.2 ft-kips
Total, beams	=	224.6 ft-kips
Total	=	245.6 ft-kips

*Siess, C. P. and Newmark, N. M., "Rational Analysis and Design of Two-Way Concrete Slabs," ACI JOURNAL, Dec. 1948, *Proc.* V. 45, p. 272.

For this span over 90 percent of the moment resistance is in the beams. Under these conditions the beam design moments are not reduced and the resulting total moment is consequently about 23 percent greater than the static moment of 200.0 ft-kips. Again, this relation is in agreement with that found in Fig. 3c. These results seem to be conservative enough.

Hess's request for a more general design procedure applicable to concentrated loads and continuity with one-way slabs will be very difficult to provide. The moment distribution procedure used by the writers in the development of the proposed specification is a step in the desired direction. However, it does not seem reasonable to expect that the analysis and design of slabs can be reduced to a procedure that is both general and simple. Simple procedures, like the one proposed in the paper, must be hedged in with numerous limitations; this is the price that must be paid for simplicity. On the other hand, methods of analysis that are general enough to consider concentrated loads, adjacent one-way slabs, etc., probably cannot be made simple enough to be acceptable in the ACI Code.

The theoretical basis for the distribution procedure has been described in some detail in Bulletin 385 of the University of Illinois Engineering Experiment Station, "Moments in Two-Way Concrete Floor Slabs."

Ferguson objects to the procedure proposed for the design of the beams, on the basis that the reduction factor Q that is applied to the beam moments in certain cases puts these beams in a class different from all other beams. It should be noted that the reduction in beam moment applies only to those beams that form an integral and essential part of the two-way slab and beam system. For those cases where the structure approaches a one-way slab, at about $b/a = 0.5$, the beams in the long span are designed the same as any other beams, since they alone carry almost all of the moment in this direction.

For the more usual proportions of panels, the writers have preferred to think of the beam and slab as a composite unit and one which compares in its general behavior with the flat slab. The consideration of redistribution of moments, or "limit design," has long been an accepted principle in the design of flat slabs. This concept provides the justification for designing flat slabs for the favorable condition of all panels loaded; it is even believed by some that it furnishes the justification for using a design moment 28 percent less than the static moment. In the belief that the flat slab and the two-way slab-and-beam types of construction are not fundamentally different in behavior or ultimate efficiency, the writers have attempted in the proposed specification to put the design of these two types of structures on a more nearly comparable basis. This has been accomplished by basing the moment coefficients, for both the beam and the slab, on the total static moment for all panels loaded. The total moment thus required is distributed to the positive and negative moment sections of the slab and beams approximately in proportion to the maximum moments that would exist at these sections for certain possible patterns of loading. It is only this apportion-

ment of moment that requires consideration of limit design or redistribution of moments.

In providing for a total moment equal to that for all panels loaded, the writers have not gone so far as to reduce this moment by 28 percent as was done for flat slabs. It was not considered desirable to go this far, partly because the test data for two-way slabs are not nearly as extensive as for flat slabs, and partly because it was felt that much of the superiority of the flat slab structure was due to the action of the slab which constitutes only a part of the two-way slab and beam system. It might be thought then that if this is the case, a reduction should have been made in the slab moments but not in the beam moments. This reasoning, however, is fallacious, since it would lead to a structure in which a larger portion of the resistance is assigned to the least efficient component, the beams.

Many factors are involved in the

Inspection of Building Construction*

By LEONARD E. DUNLAP†

SYNOPSIS

From the viewpoint of an architect, inspection on a job involving architectural and structural concrete encompasses a number of factors. Accurate plans and specifications, testing and developing the concrete mix design, proper erection of forms and placement of concrete, and systematic curing require close attention by a competent superintendent and qualified inspectors.

These remarks may extend beyond the scope of the term "inspection" because other factors must also be considered in securing the best possible results in the construction of any building or architectural structure. Considering the subject from the viewpoint of an architect, particularly where architectural concrete as well as structural concrete figure in the design, a somewhat broader range of attention is required than that which is necessarily called for in respect to inspection and testing of materials and inspections concerned primarily with structural soundness.

FACTORS INFLUENCING INSPECTION

Among these factors, the author recognizes not only those which are obviously essential, but also those which necessarily have a strong bearing upon the degree of success attained in the construction of any building. Briefly, these are as follows:

(1) A complete and accurately prepared set of drawings, fully detailed, and specifications covering all items of the work and fully describing all materials and construction processes. Documents so prepared will contribute to better performance by the contractor and to a more cooperative spirit between all parties concerned, which is so vitally necessary in any construction operation.

Documents carefully drawn will also provide the architect's and engineer's representative, and such other inspectors as may be assigned to

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†Member American Concrete Institute, Carr and Wright, Inc., Chicago, Ill.

the work, with a proper measure of authentic support in event contentions arise over any contract requirement, or in case any portion of the work deviates from that which is indicated or specified, and becomes subject to correction or rejection.

(2) The assignment of a competent and experienced superintendent to the job, to represent the architect and engineer in the field, and who shall qualify and act not only as inspector, but also assume the responsibility of supervising the activities of assistant inspectors, in case the magnitude of the work or character of operations involved require or justify such service. In view of the many trades of various nature engaged upon the average building project and the absolute necessity of intelligent coordination of the work of these trades on the part of the general contractor, it is imperative that the architect's and engineer's field superintendent give due attention to following up such coordination. This will facilitate general progress and provide the proper integration of the variety of elements involved in a construction project.

In addition to the usual qualifications required of inspectors, an assignment of this kind calls for a man possessing a well grounded over-all knowledge of construction operations based upon field experience, a knowledge of materials and their architectural and structural applications as proposed for the building project in hand, and a general knowledge and appreciation of the problems involved in the mechanical and electrical installations, which are usually many and sometimes complex. To such technical ability must be added the ability to conduct all other necessary field transactions, including the checking of contractor's estimated values of completed portions of the work for accounting and payment purposes; daily and weekly reports recording construction progress and calling attention to such conditions as may arise and require authentic interpretation or adjustment by the home office; and to transact other business or personal matters generally current in the conduct of the field office.

Finally, and by no means least important, the assignment calls for a man of good judgment who realizes the limitations of his authority; one who knows that, while having authority to require compliance with contract documents, he is under no circumstances to take any action at any time which might be construed as infringing upon any rightful privileges of the contractor. Nor must he overlook the fact that matters of procedure in construction operations are the contractor's sole responsibility and that any directions given the contractor affecting procedure must be advisory only, except in cases where specifications otherwise provide. Furthermore, the sum of a good field superintendent's personal qualities must include the ability to command respect and a good measure of diplomacy and tact in the handling of any situation

which may involve personal or business relationships between the various parties concerned with the project. He should have a mind always directed toward a fair and impartial disposition of questions affecting the work which may arise, and which fall within his authority.

(3) Following up these general thoughts concerning supervision, some remarks follow regarding our own general practice in respect to inspection affecting certain specific details which we consider of prime importance.

Materials and mix

We have always found it desirable, and in all cases specify the employment of a reputable testing laboratory to test the cement and the available concrete aggregates, and to make tests and develop a concrete mix design which will meet the requirements of our specifications. The same agency is also engaged to assign a qualified inspector who shall be responsible for seeing that proper equipment is used at the plant or site and proper methods followed to secure the approved mix, which shall be subject to further laboratory tests on cylinders taken from concrete placed as the job progresses. Too much emphasis cannot be placed upon the importance of utilizing this specialized testing and inspection service. In fact, any inclination to disregard such procedure should in the author's opinion, be earnestly discouraged. To assure an absolutely impartial and unprejudiced service of this kind, our specifications require the employment of a laboratory and inspection engineering agency by the owner and at his direct expense.

Forms

The structural requirements and character of contact surfaces of formwork are as a general rule completely set forth under our specifications and the responsibility for accurate conformance to such requirements rests with the contractor. Nevertheless, we require our field representative to see that such requirements are correctly understood and that forms are substantially built and maintained in stable and accurate position throughout the process of installing steel reinforcement, concrete and until concrete has safely set.

Placing of concrete

No element in the entire cycle of concrete production requires more care than the ultimate placing of concrete at final points of deposit, and the inspector must assure himself that the contractor has all forms in proper shape, thoroughly cleaned, wetted and otherwise prepared, and that all steel reinforcement, inserts and built-in items are in proper position before any placing operations are begun. He shall insist that all forms be provided with openings to facilitate cleaning and drainage of excess moisture where they are required.

Placing of concrete is greatly simplified with a well-designed mix delivered to the point of deposit with proper consistency. But even in such cases, care must be exercised to prevent segregation and to see that material flows properly into all corners and angles of forms and around all reinforcement and built-in fixtures. Constant and continuous supervision is essential to secure proper and complete filling of formwork and to prevent the rather common practice of depositing continuously at one point, depending upon flow to reach distant points. Such practice causes segregation of water and fine aggregates from the rest of the mass and the inspector must keep a watchful eye to prevent such procedure. There are dangers in excess tamping and puddling, but for all practical purposes such excesses are not likely to happen. The real danger is in not puddling enough against forms to eliminate the possibility of honeycomb.

Regarding the "build-up" of water in forms, especially in wall forms, which results from increased pressure on fresh material cast in lower portions of the wall, rather than drain this water off as some builders do, the writer prefers to dry up the mix for the top two or three feet of such walls, enough to offset the water. This procedure of drying up the mix as the top of the wall is approached will also eliminate to a great extent the forming of laitance.

Expansion and contraction joints in structures, and also control joints in architectural wall surfaces, are given our special attention and usually are fairly well established and called for on our drawings and the character of their construction fully detailed. Accordingly, the supervising and inspection agents are charged simply with the responsibility for the contractor's compliance. The matter of construction joints cannot always be determined beforehand, and in this case the field representatives must act to meet the situation as it arises. If conditions are unusual and require special handling, the field man has his instructions to refer the question to the home office.

Curing

One of the most difficult jobs that beset the inspector of concrete work is to secure the proper curing of concrete. We have found it necessary to make this important operation subject to a requirement inserted in our specifications, to the effect that the contractor shall assign one or more men as needed exclusively to the operation of curing, and that these men must be on hand each and every day for the period specified including Saturdays, Sundays and holidays if any such intervene during the curing period. Many curing methods have been developed, but whatever method is specified, or otherwise determined and approved, it must be carried out strictly according to instructions given by our office.

Discussion of a paper by Leonard E. Dunlap:

Inspection of Building Construction*

By JOSEPH BAYER, CARL E. V. SWANSON and S. J. WARBERG

By JOSEPH BAYER†

The writer represents a testing laboratory, and the pertinent remarks made about the inspector are very much our problem. We are generally employed by the contractor, and the answer to finding a good inspector is definitely related to the question of how much is being paid.

There are three agencies interested in the inspection: the owner, the architect and/or engineer and the contractor. Which should pay the laboratory directly? It would seem that since the quality of the work is the direct responsibility of the architect, it should be he to whom the laboratory should be responsible and by whom the laboratory shall be paid. Strangely enough the agency whose prime responsibility is to get the job done has generally been the employer of the laboratory. Let those who are responsible for quality, namely the architect, hire the laboratory.

As long as a commercial laboratory is subject to competitive bidding, the laboratory will find it is difficult to hire competent inspectors. The qualifications the inspector should possess have been spoken of at length. It is agreed that such men are hard to find and deserve remuneration commensurate with their abilities. But as long as there is competitive bidding, there is no question at all in the writer's mind that the man who is going to bid the least will have to pay his men the least, and therefore get the poorest job. Responsible laboratories have met this situation by not attempting to compete, but by quoting a fee which they know will enable them to hire competent men and at the same time enable them to continue in business.

It must come to a point where competitive bids for commercial laboratories must cease. It would seem to the writer that the architect and the engineer would be the men to see that this is done. They might enlist the help of professional engineering and architectural societies, to insist continually upon less and less competitive bids—not to the point of granting favors, of

*ACI Journal, Apr. 1950, *Proc.* V. 46, p. 609. Disc. 46-39 is a part of copyrighted Journal of the American Concrete Institute, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Bowser-Morner Laboratories, Dayton, Ohio.

course, but to assure that the work is done properly. The laboratory supplying the inspection for the architect and/or the engineer needs to be paid on a basis of competence, not of competitive bids.

We have another problem closely allied to competent inspection, namely the use of terms in advertising which imply competent inspection. For example, in some localities many ready-mixed companies advertise "certified concrete." As such, the impression is given that such certification is granted by disinterested competent agencies. Such is far from the case. The commercial laboratory may make tests once a year, or once every two years. Again we need the help of the architect or engineer to insist that such advertising be backed by competent testing inspection.

The main point the writer wishes to make is that the payment made to a commercial laboratory to supply inspectors is not a question of efficiency. There is no way to make money upon efficiency in commercial inspection. A man has to be out on the job just so long. The quality of his work is measured by his competence.

Therefore, to accomplish better inspection it is necessary to realize that the absence of competitive bidding would mean a higher rate but better inspection.

By CARL E. V. SWANSON*

We speak of inspectors and their competence. How do we get competent inspectors? They don't "just grow" like Topsy.

The ideal inspector should know more about the requirements of the plans and specifications than any of the craftsmen involved; he should know how, where and when the materials and labor of all the crafts should be applied; he should know what was intended by the architect for all portions of the work not fully covered by the plans and specifications; and last but not least, he must be able to get the work done without extras.

The writer has known competent inspectors. They were trained by the architects, engineers, contractors and craftsmen they worked for and with. Their value on the job consisted of not only the "know-how" of construction in obtaining good results in the finished product but also, from past mistakes, had accumulated a wealth of knowledge of *what not to do* in order to forestall poor results.

The writer cannot agree with the suggestion that stringent penalty clauses will insure better results in construction. The standard forms of the contract between owner and contractor adequately cover cases of faulty workmanship and materials. As an architect, the writer would be loath to imply, through further use of penalty clauses, that the contractor to whom construction is entrusted is not competent to produce the results desired. Our office does not invite bids from incompetent firms, and that is a right we insist upon.

It has been faith in the honesty and integrity of others that has made this nation; and confidence in someone else's ability to produce that has

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made great engineering projects successful.

To tear out and replace faulty work is a common penalty. The architect who sent the writer on his first job as an inspector impressed on him the responsibility to stay ahead of the construction and forestall errors. In the disposition of controversial matters he said, "Whenever you know you are right, make your decisions stick. But remember this, sometimes when you are the most right you can be the most wrong." In other words, when a faulty piece of workmanship appears on the job, through misunderstanding, poor judgment or hard luck, it is often times more damaging to the completed project to tear out and replace than to try to get around the error by a change in detail. This calls for cooperation between the architect and contractor, and, if the change required is of too serious a nature, it must be referred to the owner. Our office has always found the contractors willing to rectify their errors in accordance with our recommendations and, as a consequence, we feel that added penalty threats are unnecessary.

By S. J. WARBERG*

The inspector has to have specifications to go by; but if those specifications are not properly written, there isn't much that the inspector can do and particularly so if the specifications are of the same form as the law which provides that one mustn't do so and so but does not provide any penalty if one does it. Specifications should be written so that there is a definite penalty.

It may be said that the penalty would consist of having the work rejected, but in most private or industrial jobs, that is something which the owner resents because it delays the completion of the job. Therefore, there is a valid objection to condemned work; delaying the completion of a job often results in a large loss of income or return on the investment. In many cases it is impractical to do it.

The writer has talked with specification writers about this and they claim that unless there is a premium provided for performing according to specifications, there can be no penalty attached for not performing; and, since our specification writers generally write not for concrete only but for all trades, it would be a continuous headache for them. That indicates that we should get a good contract lawyer to frame our specification language to our purpose.

If one provides a definite penalty for the contractor who does not live up to the specifications, it will make him live up to them; it is money in his pocket to live up to them. The penalties should be big enough so that if he doesn't—if he infringes—the owner will deduct from his bill more money than he would save by not living up to the specifications.

That also means that the inspector should be backed up to the hilt by the owner, and he should be paid by the owner. He should not be paid by a testing laboratory that is hired by a contractor; that comes back into the contract price with a profit and handling charge added and the owner pays it, as he pays everything. The owner might as well pay it direct and avoid

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paying a profit to the contractor on that particular item; it's a saving in his pocket.

The inspector can be and should be a diplomat. He will find that when he tries to direct a workman out on the job, it is much better to tell the workman why he must do it differently—give him the underlying principles. The workman is not a "ninny;" he has intelligence and takes pride in the work he does. If the inspector tells him why a tension rod must be where tensions occurs, he can see it. The inspector can also demonstrate to him that excess water, which results in a lot of cement being washed out, is not good. There are lots of ways that the inspector can tell the workman diplomatically why he must do things in such a way and not the wrong way, without arousing any ill feeling, and it's quite important to do so.

Most of these points were brought out in a paper in the November 1941 JOURNAL.* That article also pointed definitely to the reprehensible practice of letting the contractor hire the testing laboratory. No testing laboratory likes to be beholden to a contractor for its business—it affects the honesty of its reports to the owner.

These considerations make one ask the question, "How fast is our progress along inspection lines, and aren't we progressing much faster along other lines?" We make all kinds of investigations to find out how to improve the structural qualities of concrete, but the task of insuring that we get that kind of concrete by proper specifications is lagging behind.

The inspector has to know his job, but he ought to have the proper tools to work with, and unless he can enforce his inspection by penalties—and not have to resort to condemnation, which results in delay—he is at a handicap.

When the contractor sees that by doing a thing wrongly he is definitely going to lose money, he isn't going to do anything wrong. He isn't going to do it because it doesn't pay; the only reason he'll do the other is to make money.

After all, one can buy a solid cubic yard of cement for about $40, a solid cu yd of gravel or sand for about $6, a solid cubic yard of water for 30 cents. Now, if he's going to cheapen the mixture of those three, what is he going to use more of and what is he going to put in less of? It stands to reason that he isn't going to put in more cement.

*Warberg, S. J., "Concrete Specifications and Water Content of Concrete," ACI JOURNAL, Nov. 1941, *Proc.* V. 38, p. 169.

A report on the nature and potential reactivity of aggregates encountered in Australia.

Australian Aggregates and Cements in Relation to Cement-Aggregate Reaction*

By A. R. ALDERMAN, A. J. GASKIN, R. H. JONES and H. E. VIVIAN†

SYNOPSIS

A wide variety of Australian aggregates and cements has been examined with a view to estimating the possible incidence of expansive reaction in concrete made from these materials.

Mortar bars were observed over periods up to two years and the results correlated with petrographic examination of the aggregates and chemical composition of the cements. This correlation has shown that in most cases the potential reactivity can be assessed by petrographic examination but that doubtful material requires supplementary mortar tests.

This paper summarizes investigations which have been described in greater detail elsewhere.‡ The work was planned to serve a double purpose; to investigate the probable incidence of cement-aggregate reaction in Australia, and to provide a basis on which more fundamental investigations of the mechanism of the reaction could be developed.

Altogether, 67 aggregates were used in the investigation. They included materials used extensively in many parts of the continent and were of wide petrographic range. The 16 cements which were used were obtained from 10 Australian manufacturers.

As no shorter method commended itself, the work of T. E. Stanton§ was followed in the making of mortar bars, their storage in moist air in sealed containers, and their periodical examination and measurement.

Except when samples were limited, each aggregate was tested with at least four cements of varying alkali content. As most Australian cements are sold only in the State in which they are manufactured, it was possible to arrange the cement-aggregate combinations so that

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†Officers of the Division of Industrial Chemistry, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia.

‡Council for Scientific and Industrial Research (Australia), *Bulletin* 229, 1947, pp. 7-46.

§*Proc.*, A. S. C. E., V. 66, 1940, p. 1781.

each aggregate was tested with all the cements with which it was likely to be used. Two cements, one high and the other low in total alkalies, were used with each aggregate and thus provided standards for comparing aggregates. Two aggregates, one a reactive siliceous magnesiam limestone from California, the other an inert quartz sand from Leighton Buzzard, England, were used with each cement and so provided standards for comparing the cements.

Since it was desired to investigate the reactivity of Australian aggregates with commercial cements, no special treatment was given to the cements used for making test bars. The total alkali content ($Na_2O + 0.658\ K_2O$) of the cements ranged from 0.12 to 1.09 percent.

All aggregates were examined petrographically, particular attention being given to the recognition of opaline silica. If the examination of thin sections under the microscope suggested that opal might be present, a sample of the aggregate was crushed and sieved (-72 mesh), the fine material was suspended in a bromoform-alcohol mixture of density 2.36 and centrifuged. The light fraction, which was usually small and contained nearly all the opal, was then collected, washed with alcohol, dried and weighed. Confirmation of the presence of opal was obtained by microscopic examination of this preparation immersed in a liquid of suitable refractive index (*e.g.*, 1.47). As little as 0.01 percent of opal could sometimes be estimated by these means.

In some aggregates which showed "delayed" reaction with high-alkali cement, the reactive component which was cryptocrystalline had an average refractive index (1.55) which was higher than the corresponding index of either chalcedony (1.53) or opal (less than 1.46). This material appeared to be cryptocrystalline quartz, and although fine opal could conceivably be present as intergrowths, careful microscopic examinations did not disclose it.

Conventional methods were used in the preparation of aggregates and in the fabrication, storage and measurement of mortar bars. Storage of the sealed containers at room temperature was adopted because of convenience and approximation to service conditions. The behavior of control bars, made with standard nonreactive sand, showed that length variations due to daily change in temperature and consequent readjustment of the air in the container to moisture-saturation were insignificant compared with those produced by expansive reaction. Seasonal changes in temperature, however, have somewhat larger effects. Although the end results may be similar, the initial rate of expansive reaction of mortar bars fabricated during the summer is generally greater than that of bars made in cold weather.

In addition to the test bars, small mortar blocks were made of each aggregate with a high-alkali cement and a low-alkali cement. These

blocks, stored under conditions similar to those of the larger test bars, were broken periodically and their broken surfaces examined with a microscope. Spots of wet gel-like material surrounding the aggregate particles generally indicated that the aggregate contained reactive components even though the mortar may not have expanded significantly.

Results and discussion

The results of the many thousands of measurements and observations of mortar bars over a period of two years show that most bars expanded slightly under the conditions of storage (in moist air, in sealed containers, at room temperature). In some instances, the expansions have reached 0.05 percent which, in the absence of other signs of reaction, can be regarded as normal. When, in the same period, expansion exceeds 0.05 percent the aggregate or the cement-aggregate combination requires further examination. Expansions of 0.1 percent are accompanied by cracking and can be regarded as dangerous.

When some reactive aggregates are used with low-alkali cements, expansion is negligible. However, if aggregates are very reactive, such as those containing suitable amounts of opal, even low-alkali cements produce expansions which may develop only after one or two years. In such cases it seems that the limitation of alkalies in cement to even a very low figure will not ensure permanant immunity from expansive reaction. At present there seems to be no alternative but to condemn such aggregates for use with any cement.

Summarized briefly, the methods used to assess the reactivity of aggregates were: petrographic examination of the aggregate in order to recognize possible reactive components; macroscopic and microscopic examination of mortars containing the aggregate for the purpose of recognizing gel-spots, cracks and other signs of reactivity; and determination of expansion by linear measurement of such mortars.

In correlating petrographic type with reactivity as aggregate, useful generalizations were obtained for some types of aggregates. The varied origins of natural sands and gravels makes such correlation difficult. Of the more than 30 such aggregates examined, half gave no signs of reactivity, somewhat less than half gave slight signs of reactivity and two or three caused significant though small expansions. These facts suggest that close examination of such materials is desirable. When gravel, or in fact any rock, comes from areas where surface silicification has been active it should be examined carefully as opal or cryptocrystalline silica may be present. Consolidated siliceous sediments such as sandstones and quartzites have characteristics very similar to those of unconsolidated sands and gravels and the probability of their being reactive seems to be about the same.

Volcanic or hypabyssal rocks of the acid or intermediate groups (rhyolite, trachyte, porphyry, etc.) should in general be examined carefully. Reactivity of these rocks is associated with the presence of glass. The 12 samples of volcanic and hypabyssal rocks of the basic group which were examined in this investigation were unweathered and free from secondary silicification. None of them gave any signs of reactivity, which suggests that basalts and dolerites are particularly safe aggregates. It must be remembered that rocks of this type, especially when vesicular, are subject to a type of silicification in which opal is produced in quantity. Weathering and other secondary changes are major factors in determining the potential reactivity of rock when used as an aggregate. This generalization will apply to most rocks. Only three granites were included in this investigation, and although they showed slight signs of reactivity, there seems little evidence to suspect that granite when unweathered is not a safe aggregate.

Summary and conclusions

Careful observations of mortar bars for two years disclose that very few aggregates promote significant expansions and although many showed that they contain traces of reactive constituents the great majority can be regarded as safe in this respect. However, the wide geographical distribution and varied petrographic nature of the relatively few dangerous aggregates suggest that aggregates should always be carefully examined before use.

Support is given to published evidence that the presence of opal in aggregates must be regarded as dangerous and that the use of acid and intermediate volcanic rocks must be regarded with suspicion. In Australia, aggregates from the central arid regions, whether rocks in situ or unconsolidated sands or gravels, may be dangerous owing to the possible presence of opal. In several rocks, cryptocrystalline quartz appeared to cause a dangerous reaction.

Correlation of the behavior of mortars with the petrographic character of their aggregates suggests that much can be learned from petrographic study. In many cases this should be sufficient to evaluate an aggregate.

Although the use of low-alkali cement with a reactive aggregate may modify or delay deterioration, the evidence tends to show that such aggregates are undesirable for use with any cement.

Acknowledgments

For samples used in this investigation, we are indebted to T. E. Stanton and the California Division of Highways, the South Australian Dept. of Mines and others too numerous to mention individually. We are also grateful for the invaluable cooperation and financial assistance of the Cement and Concrete Assn. of Australia.

The work described in this paper was part of the research program of the Division of Industrial Chemistry of the Commonwealth Scientific and Industrial Research Organization, Australia.

Unique theories are advanced which will be of interest to American readers.

Some Australian Studies on Cement-Aggregate Reaction in Mortar*

By H. E. VIVIAN†

SYNOPSIS

This paper summarizes briefly some of the papers on cement-aggregate reaction which have been published by the Commonwealth Scientific and Industrial Research Organization in Australia. These papers deal with four aspects of mortar expansion; the change in mortar tensile strength as expansion occurs, the effects on mortar expansion of alkali mobility, of void space in the mortar and of different storage conditions.

The early work of Stanton‡ and others on the problem of cement-aggregate reaction showed that there was sometimes an abnormally high expansion in mortars and concretes made with high-alkali cements and certain aggregates that could react with alkalies liberated from the cement during hydration. These investigations showed that aggregates containing opal or certain types of volcanic rock were especially subject to this reaction.

Much experimental work has also been done to determine the conditions necessary to produce aggregate reaction and mortar expansion. It has been shown that moisture conditions of storage, storage temperature, the proportion of cement to aggregate in the mortar, the quantity of reactive material in the aggregate, and the size of the reactive particles all affect the development and magnitude of mortar expansion.

The work summarized in this paper formed part of a research program planned to investigate the mechanism of expansive reaction. Much of this work has been done with a high-alkali cement containing 0.81 percent K_2O and 0.45 percent Na_2O; where necessary, cements of lower alkali content have been used. All the reactive aggregates used were mixtures of an opal-containing rock and an inert quartz sand. No attempt

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‡*Proc.*, A. S. C. E., V. 66, 1940, p. 1781.

is made in this summary to give details of the experimental work which can be found in the original papers.* The general conclusions are discussed briefly as follows: (1) changes in the tensile strength of mortar, (2) the effect on expansion of void space in the mortar, (3) alkali mobility in the hardened mortar, and (4) the effect on expansion of different storage conditions. The implications of this work on the mechanism of mortar expansion are mentioned briefly.

Changes in tensile strength of mortar

Examination of mortar which has expanded abnormally always reveals the presence of macroscopic cracks. Such cracks are associated with reacted aggregate particles and reduce the tensile strength of the mortar by as much as 75 percent in certain instances. Observation has shown that fine, short cracks are produced in the mortar before expansion becomes abnormal and before the tensile strength is reduced significantly. Tensile strength reduction became apparent before expansion commenced. The time required before the onset of tensile strength reduction and of mortar expansion depended on the alkali content of the cement. The experiments suggest that, over a long period of time, mortars made with low-alkali cement and reactive aggregate will expand and crack sufficiently to reduce tensile strength significantly.

This work has shown that mortar expansion depends on the development and widening of cracks and that the cracks are not completely filled with the reaction product. Careful examination of expanding mortars shows that the reaction product is in the gel condition. The reaction product remains in the gel condition for a considerable time but eventually tends to become fluid and ceases playing an active part in expanding the mortar.

Effect on expansion of void space in the mortar

The degree of compaction of mortar plays a large part in controlling expansion. If the mortar is not fully compacted, large air-filled void spaces are present and these have a marked effect on the transmission of the forces from the swelling reactive particles to the mortar.

By decreasing the proportion of cement to aggregate in the mortar the amount of void space was increased. A similar effect was obtained by decreasing the amount of mixing water. Mortars which contained similar amounts of void space but different proportions of water, cement and aggregate were found to give similar expansion results. It was found that when the amount of void space in the mortar exceeded 7-8 percent by volume, the mortar was not expanded abnormally; when the amount of void space was less than 7-8 percent by volume, mortar expansion was abnormally large. In both instances the reactive aggregate was attacked

*Council for Scientific Industrial Research (Australia), *Bulletin* 229, 1947, p. 47-77. *Journal*, Council for Scientific and Industrial Research (Australia), V. 20, 1947, p. 585.

and had expanded. In the more compact mortars the reacting particles exerted swelling pressures on the mortar and caused cracking and expansion, whereas, in the less compact mortars the reacting particles expanded into voids and the mortar itself was not seriously cracked nor expanded. Thus aggregate expansion causes large cracks and mortar expansion only when the aggregate swells against a restraining barrier.

The effect of void space on changes in tensile strength due to aggregate expansion was followed over a period of some months. The tensile strength of compact mortars containing reactive aggregate declined by about 75 percent whereas there was no significant decline for mortars which contained large amounts of void space. Long, wide cracks were developed only in compact mortars containing reactive aggregate and high-alkali cement; any cracks observed in the porous mortars which contained reactive aggregate were very short.

This evidence indicates the importance of void space in limiting expansion and severe cracking. It also suggests that reactive aggregates that are porous (*e.g.*, pumice) may not cause abnormal expansion since they contain voids in which the reaction product can be accommodated without exerting disruptive swelling pressures on the mortar.

Mobility of alkalies in hardened mortar

The dependence of the commencement of expansion and the reduction of tensile strength on the cement-alkali content, suggests that alkali mobility in the hardened mortar may be an important factor in both aggregate attack and mortar expansion. It can be assumed that alkalies must diffuse through the solution in the mortar to the reactive aggregate particles to cause their severe attack.

The mobility of alkalies in hardened mortar was demonstrated by the use of "composite" test bars. One type of composite test bar consisted of alternate sections of low-alkali cement with reactive aggregate and high-alkali cement with nonreactive aggregate. In such bars the reactive aggregate was not in direct contact with high-alkali cement. After some weeks storage in moist air, the reactive particles nearest to the high-alkali cement began to react and the bars started expanding. By varying the arrangement and composition of the different sections in the composite bars it was shown that alkalies could migrate through considerable thicknesses of mortar. It is reasonable to assume that, provided moisture conditions remain suitable, sufficient alkali migration and consequent expansion will eventually occur in mortars made from low-alkali cement and reactive aggregate. Thus the adoption of an arbitrary limit for the alkali content of cement for use with reactive aggregates may be inadequate, since it may not prevent serious reaction and abnormal expansion of mortar and concrete but only delay it.

Effect on expansion of different storage conditions

One surprising feature of the expansions of composite test bars was the magnitude of their expansions when these were calculated as a percentage of the total length of reacted mortar, not the total length of the mortar bar. The calculated expansions of bars consisting of sections containing reactive aggregate alternating with sections containing non-reactive aggregate were generally much greater than the expansions of uniform bars containing reactive aggregate. These observations suggested that the amounts of water or alkalies, or both, available to the reactive aggregate in composite bars were increased and had an important effect on mortar expansion. Later evidence shows that water is as essential for mortar expansion as are reactive aggregate and alkalies.

Experiments made on the effects of storage conditions on the expansion and tensile strength changes of mortar have shown how they can modify alkali and moisture conditions inside test specimens. Mortars made with high-alkali cement and reactive aggregate and stored in dry air did not expand; they actually contracted. However they were cracked and their tensile strengths were generally reduced to about 50 percent of similar control specimens. When this behavior is compared with that of similar mortars stored in moist air (positive expansions of more than 1 percent and tensile strength reductions of 75-80 percent) it can be seen that water must be available for absorption by reacted particles before either abnormal expansion or severe cracking can occur.

Mortars of the same composition stored in water did not undergo large expansions; those stored in alkali hydroxide solution, however, expanded abnormally. These differences are due to the diffusion of alkalies from the mortar into the surrounding storage solution when the latter contains little or no alkalies. The presence of the large quantity of water available for absorption by the reaction product tends to accelerate its change from the gel to sol condition and this also reduces mortar expansion. Neither alkali content nor water was limiting when mortar was stored in an alkali solution and consequently expansion was large. The tensile strengths of mortar stored either in water or in alkali solution were reduced to about 45 percent of the control mortar specimens. This indicates that in both instances the mortars containing reactive aggregate are severely cracked.

Conclusions

Mortar containing high-alkali cement and reactive aggregate is expanded by a reaction product that is present in the gel condition. The swelling aggregate particles produce cracks, which are not completely filled by the reaction product, unless unusually large quantities of water are available for absorption. Observation has shown that the reacting

particles do not become soft and readily deformable until considerable time has elapsed after expansion begins. The shape of the expansion-time curves of mortar bars suggests that, as the expanding reaction product becomes fluid and deformable, mortar expansion diminishes.

No evidence has been found to suggest that the reaction product is confined to its site of formation by a semi-permeable membrane. It is capable of flowing or is forced along cracks and when large voids are present at or near reacting particles the reaction product will move into them. It seems reasonable to assume that mortar is expanded by swelling gels, not by solutions.

The mobility of alkalies in hardened mortar suggests that the aggregate reaction process can occur over a long period. Thus even low-alkali cements, if given sufficient time, may cause expansion and cracking.

Acknowledgment

The work described in this paper was part of the research program of the Division of Industrial Chemistry of the Commonwealth Scientific and Industrial Research Organization, Australia.

Carbon dioxide can be used in preventing spotting and expansion of cement mortars due to alkali-aggregate reaction.

Carbon Dioxide and the Cement-Aggregate Reaction*

By A. J. GASKIN†

SYNOPSIS

Spotting and expansion of cement mortars due to alkali-aggregate reaction can be prevented by treatment of the set mortar with carbon dioxide. Active alkali hydroxides, produced by hydrolysis of cement particles, are thereby converted to carbonates, which are inactive in contact with most aggregates. Sufficient "protective" calcium carbonate can be produced throughout the mortar to confer permanent immunity from spotting and expansion, but no appreciable improvement in tensile strength could be obtained. Carbonated mortars were found to have a high degree of dimensional stability.

It has been shown by Bean and Tregoning‡ that opaline aggregates are attacked by alkalies only when alkali hydroxides are present in solution, while alkali carbonate and sulfate solutions are inactive. Stanton, et al,§ found that certain pozzolanic materials effectively removed alkali hydroxides from the solutions present in cement mortars, thus preventing alkali-aggregate reaction from developing.

In the present investigation, reported in greater detail elsewhere,¶ attempts were made to prevent alkali-aggregate reaction by immobilizing, in the form of carbonates or sulfates, the alkalies in cement mortars.

Preliminary tests showed that it was not possible to "carbonate" mortars successfully by introducing soluble bicarbonates into the mixing water, nor was it practicable to produce alkali sulfates in the set mortars by treatment with gaseous sulfur dioxide. It was found, however, that saturation of set mortar with gaseous carbon dioxide was effective in preventing alkali-aggregate reaction from causing noticeable spotting and expansion.

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†Officer of the Division of Industrial Chemistry, Commonwealth Scientific and Industrial Research Organization, Melbourne, Australia.

‡Bean, L. and Tregoning, J. J., ACI JOURNAL, Sept. 1944, *Proc.* V. 41, pp. 37-51.

§Stanton, T. E., Porter, O. J., Meder, L. C. and Nicol, A., ACI JOURNAL, Jan. 1942, *Proc.* V. 38, p. 209

¶Gaskin, A. J., Council for Scientific and Industrial Research (Australia), *Bulletin* 229, 1947, p. 78.

The concentration of carbon dioxide necessary was found by experiment to be at least 90 percent by volume, in a carbon dioxide-moist air mixture, used at normal atmospheric pressure. Mortars required immersion in such an atmosphere for a period of 28 days after initial set, during which time the mortar generally absorbed about 2.6 percent of its weight of carbon dioxide. After such treatment, mortars containing reactive aggregates were stored in moist air for the duration of the test (12 months) without showing any signs of spotting or expansion.

Carbon dioxide appears to have a two-fold action on mortars. Alkali hydroxides are converted to inactive carbonates while the mortar is kept saturated with the carbon dioxide-moist air atmosphere. At the same time, carbon dioxide reacts with calcium hydroxide solutions produced in the mortar by hydrolysis of cement, calcium carbonate being precipitated throughout the mortar.

If carbonation is continued for about one month, sufficient calcium carbonate is deposited around the reactive aggregate particles to isolate these particles inside impermeable protective shells. If the mortar is removed from the carbon dioxide atmosphere and stored in moist air, these shells prevent further contact between the reactive aggregate and any alkali hydroxide solutions that may be regenerated by reaction between the alkali carbonates still present in the mortar solutions and calcium hydroxide formed by later hydrolysis of cement grains.

Since the experiments were made under very moist conditions, the rate of carbonation was comparatively slow. The aggregate grains appeared to be attacked at a greater rate than it was found possible to effect carbonation throughout the mortar. The tensile strength of the mortar was thus lowered by alkali-aggregate reaction in the usual manner. Tensile strength reduction can apparently be brought about by a small amount of alkali-aggregate reaction, whereas spotting and expansion require the production of comparatively large quantities of gel.

Carbonation had no effect on the tensile strength of test pieces containing no reactive aggregate, and no case of "carbonation crazing" was observed in any specimen during the investigation.

An interesting feature of the experiments was the pronounced dimensional stability of all mortar bars subjected to carbonation. A slight initial expansion (up to 0.01 percent) was invariably associated with the first stages of carbonation, after which the carbonated bars showed deviations of only ± 0.003 percent about a mean length over a period of a year. During the same time, normal air-cured bars, containing no reactive aggregate, showed length increments of up to 0.04 percent while stored in moist air.

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The work described in this paper was part of the research program of the Division of Industrial Chemistry of the Commonwealth Scientific and Research Organization, Australia.

Precast joist tests are described with bar spacing less than that required by the ACI Code.

Spacing of Moment Bars in Precast Joists*

By F. N. MENEFEE† and H. L. KINNIER‡

SYNOPSIS

Spacing of moment bars in precast reinforced concrete joists in some cases has been as small as $\frac{3}{16}$ in. although it is probable that most manufacturers use $\frac{7}{8}$-in. shear or diagonal tension bars with corresponding spacing of moment bars.

ACI Building Code places a one in. minimum on spacing. Undoubtedly, the regulations were written with monolithic concrete rather than light precast joists in mind. Spacing has no particular significance if all the requirements for bond are met independently of it.

In a properly designed precast joist, theoretical computations show that bond stresses are always less in proportion to the maximum allowed than are any of the other stresses.

The tests herein reported were an attempt to determine whether the artificial reduction of bond area, up to 30 percent, had any effect on the over-all strength of the joist, and to give some indication as to whether $\frac{7}{8}$-in. spacing of moment bars introduced a controlling weakness.

While the results on 14 joists, all of the same dimensions, could hardly produce conclusive evidence, the results, along with the service record of precast joists, indicate the $\frac{7}{8}$-in. spacing of moment bars, with $\frac{3}{4}$-in. maximum size aggregate and with all other requirements of the Building Code met, will give a joist which will fail otherwise than in bond, from which it appears that the present one in. minimum in the current Building Code should be thoroughly studied with a view toward modification for precast joists, with well anchored reinforcement—if opinion and test justify such a change.

HISTORY

The idea of precast reinforced concrete structural members is not new. As early as 1912 one of the authors assisted in the design of a grandstand where the horizontal girders between columns and the slant girders for supporting the deck were to be precast on the job and hoisted

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into place. The design was rejected in favor of a monolithic type which today has two large cracks from top to bottom of the horizontal girders and some cracks in the deck girders. Today several concerns make a specialty of producing heavy precast members for industrial buildings.

In the middle thirties, concrete products manufacturers became interested in the development of means for making relatively light reinforced concrete joists, at a central concrete products plant, for delivery to building sites after curing. With the demand for housing, the expansion of apartment house building and the high cost of forming on the job for small dimension joists, the precast reinforced concrete joist was introduced and is now firmly established.

METHODS OF FORMING JOISTS

There are two methods of forming I-beam type joists; in each, the reinforcing is all assembled in advance; the shear rods, either slant or vertical, are welded to the moment bars top and bottom. These assemblies of the reinforcing are sometimes called "cages" (Fig. 1).

In one method of manufacture the "cage" is placed in a "pallet" or steel form, securely spaced therefrom. The concrete is placed on the pallet just behind a roller that rolls the concrete into shape under high pressure as the pallet is pulled forward under the roller. These joists are formed lying on their sides.

In the other method, the forms, usually of steel, set upright in gangs on a vibration table, so that 10 to 14 joists are formed at one time.

In either method, the placing of the reinforcing can be accurately controlled. In the tests made, that feature was particularly noticeable.

Factory made joists of this I-beam type are made in dimensions ranging from 3 x 6 in. to 4 x 14 in. Various types of aggregate are used, usually with a maximum size of $\frac{3}{8}$ in.

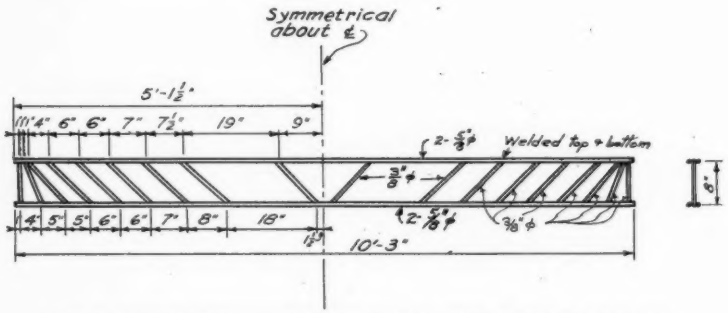

Fig. 1—Typical reinforcement "cage" for 3½ x 10 in. joists, No. J-1 to J-4

The moment bars vary from two ⅜-in. round at the bottom of the 3 x 8-in. joist, to two 1-in. round in the bottom and two 1¼-in. square in the top of the 4 x 14-in. joist. The shear rods are generally ⅜-in. round, welded in between the moment bars, making a ⅜-in. space between the moment bars. From a manufacturing standpoint this is ideal since the size of the shear bars may be kept constant and their spacing can be varied according to the need for shear reinforcing.

The Minimum Standard Requirements for Precast Floor Units (ACI 711-46) is silent on the subject of spacing of moment bars in these precast joists. Section 10(*b*) thereof reads in part as follows: "With respect to design for strength, *i.e.*, for bending moment, bond and shear stresses, all the types (of precast reinforced concrete joists) are to be designed in accord with standard reinforced concrete theory and in accord with (ACI 318-41) . . . " Accordingly, one has to go to the Building Code for a specification relative to spacing. Section 505-A thereof reads in part as follows: "If special anchorage as required in Section 903 is provided, the minimum clear distance between parallel bars shall be equal to the diameter for round bars and 1½ times the side dimension for square bars. In no case shall the clear distance between bars be less than 1 in. nor less than 1 and ⅓ times the maximum size of the coarse aggregate . . . "

From the foregoing it is plain that the precast joist manufactured as already indicated, falls without the requirements of the ACI Building Code. In spite of this, these joists are being used and are giving satisfaction. As far as the writers know, there have been no failures.

Undoubtedly the ACI Code, establishing a minimum spacing of 1 in., was written for monolithic concrete beams and girders, few of which were ever as narrow as 3 in. and where the maximum size of the coarse aggregate was seldom less than ¾ in. Furthermore, the possibility of the moment bars being carelessly placed with respect to spacing was well known at the time the minimum spacing was established.

It is well known that the bond stress times the bond area over a unit length of moment rod must equal the change in tension or compression in the moment rod per unit length and that the cross section of the concrete at the steel level must be sufficient to take the longitudinal shear set up by the change in tension in the steel.

Regardless of how the matter is approached, bond stress and complete envelopment of the steel is generally considered to be essential except in unbonded prestressed reinforced concrete and possibly in precast reinforced concrete joists of the type under investigation. The reinforcement in these joists is exceptionally well anchored because of the welded connection between web and main steel.

OBJECT OF THIS STUDY

To throw some light on the question of why joists with ⅜-in. spacing between moment bars seemingly give satisfactory results under service loading, an investigation was planned covering both theoretical considerations and actual tests.

The theoretical study started with a computation of the bond stress from the standard formula:

$$u = \frac{V}{\Sigma o j d}$$

Typical joists were chosen from the table of safe uniform loads of a reputable company manufacturing precast joists. From the dimensions, the bond stress u was computed. For instance, a joist of 8-ft span, with jd approximately 10 in., an allowable V of 4232, and with $\Sigma o = 5.5$ in., gives a bond stress, at rated allowable loading, of 77 psi whereas the allowable bond stress for special anchorage is 150 psi. This type of "cage" reinforcement was assumed to fall within the special anchorage classification. Subsequent tests proved this assumption to be correct.

Other joists gave values for u of 99, 49 and 49 psi respectively; from which it appeared that the maximum theoretical bond stress for the rated loading was about 66 percent of that permitted. It was known that the 150 psi allowable was based on a possible imperfect coverage or envelopment of the steel and that bond tests show that 3000 psi concrete will usually develop bond stresses of 2½ times or more than 150 psi.

Theoretically one could reduce the envelopment of the steel by 20 percent or more in these joists and still not exceed the 150 psi allowable bond stress. Reduction in envelopment is considered because we recognize that one of the reasons for a spacing specification is that we do not want aggregate to bridge from one bar to another and prevent full envelopment of the steel. If, however, the bond area could be reduced by 20 percent without giving a theoretical bond stress of more than 150 psi, that particular reason for requiring a 1 in. minimum would at least be subject to question.

It was thought that both methods of manufacture of precast joists with ⅜-in. spacing between the moment bars forced the concrete well in between the bars in spite of the aggregate usually running from fine to a maximum of ⅜ in. It was also thought that tests run on typical joists in which the bond area was deliberately reduced 10, 20 and 30 percent might be of value in determining the extent to which the narrow spacing might be detrimental.

PROCEDURE

Fourteen 3½ x 10-in. joists, 10 ft long, reinforced as shown in Fig. 1

were made by the rolling process. The concrete was mixed in the following proportions:

Fine Waylite	120 lb
Coarse Waylite	30 lb
Limestone screening	70 lb
Torpedo sand	60 lb

4 sacks cement with 4 to 4½ gal. of water per sack of cement.

From the standpoint of high pressure rolling this dry mixture produced an acceptable joist and probably gave higher compressive strengths than if the water ratio had been greater. The longitudinal bars were two ⅝ in. round, both top and bottom. The diagonal tension rods were ⅜ in. round. Cylinder tests on the concrete indicated it to be somewhat better than 3000 psi. Two tests on the reinforcement indicated a yield point of about 70,000 psi.

Inspection after the tests conclusively showed that the high-pressure rolling forced the aggregate in between the moment bars throughout their full length. In one sense of the word, there was complete envelopment. However, close inspection of the broken joists following the tests indicated that while the bars were fully enveloped and the aggregate was wedged firmly between the bars, actual contact between the concrete and bars due to lack of water or fines was probably not greater than 50 percent.

The loads were applied at the third points by a 200,000-lb capacity Riehle testing machine. While not an ideal procedure, two like joists were tested simultaneously, in order that this, a pilot test, could be completed in the time available. There was nothing to indicate that the conclusions would have been materially affected had the joists been tested individually.

Of the 14 joists, there were two joists made as if for regular use, *i. e.*, no artificial reduction of bond area. Four were made with strips of tape covering 1/10 of the periphery of the bottom moment bars and located on the inside surface, *i.e.*, that locality where bridging would be apt to occur. Four joists were made with tape reducing the periphery of the moment bars by 20 percent, and four more reducing it by 30 percent.

Deflections were measured midway between the supports by means of a scale taped on the side of the joist with a light strong thread kept taut by rubber bands stretched between points immediately over the supports midway between the top and bottom of the joist.

After an initial reading with no loading, a load of 1000 lb was placed on the two joists, after which the loading proceeded generally in 2000 lb increments until failure took place (Table 1).

During the loading process careful observation of the deflections and close inspection for cracks was made.

TABLE 1—TEST DATA

$V = W$ $\quad w = \frac{2W}{l} = \frac{W}{l/2}$ $\quad V = wl/2$

$M = \frac{Wl}{3}$ $\quad w = \frac{8W}{3l} = \frac{W}{3/8l}$ $\quad M = \frac{wl^2}{8}$

Joist No.	W (at failure), kips	Equiv. w (at failure)		Safety factor for w = 720 lb per ft* based on failure	
		From V	From M	From V	From M
J1N1	8.23	1690	2250	2.35	3.12
J1S1	8.23	1690	2250	2.35	3.12
J2N1	9.35	1918	2560	2.66	3.56
J2S1	9.35	1918	2560	2.66	3.56
J2N2	10.00	2050	2740	2.85	3.80
J2S2	10.00	2050	2740	2.85	3.80
J3N1	8.95	1838	2450	2.55	3.40
J3S1	8.95	1838	2450	2.55	3.40
J3N2	9.75	2000	2670	2.78	3.71
J3S2	9.75	2000	2670	2.78	3.71
J4N1	8.48	1740	2320	2.42	3.22
J4S1	8.48	1740	2320	2.42	3.22
J4N2	7.75	1590	2120	2.21	2.94
J4S2	7.75	1590	2120	2.21	2.94

*Rated uniform load in manufacturer's published tables.

In seven cases, final failure came by a right sectional fracture in the bottom steel near a weld between the moment bars and the diagonal tension bars. Some failures occurred in bearing at the ends of the joist where the bearing plates became bent over the supports, producing more or less of a line bearing instead of the 3½ x 3½ in. that was planned.

In no case was the moment at final failure less than 2¼ times the moment given by the rated uniform load per foot except where the bond area had been artificially reduced by 30 percent.

With reference to the data in Table 2, column 8 indicates the unit bond stress at first crack and the values indicated are probably as nearly correct as the formula for bond ever gives. Actual bond stresses at failure are generally greater. It is not known whether the first crack was due to failure in bond or to a failure in tension of the concrete. It was probably the latter, for the stresses in the steel shown in column 12 ran from about 14,000 to 16,000 which in turn would give higher tensile stresses in the concrete than it would normally be expected to withstand.

Column 9 indicates the theoretical bond stress at joist failure. The figures shown are a little higher than those shown in Table 7 of Bulletin 147, Iowa Engineering Experiment Station, by Gilkey, Chamberlin and Beal. Possibly at some lower figure than those indicated in column 9 the bond failed throwing the burden for beam action on the anchorage provided by the welds.

Columns 4 and 5 were computed as indicated in the explanation of Table 2, taking into consideration the fact that there will be neither longitudinal nor transverse shear stress in the cracked section. The shear resistance in a cracked section would come from the upper and lower longitudinal and diagonal steel plus that from the concrete above the top of the crack.

CONCLUSIONS

For the first series of 14 joists there is no indication that the ⅜-in. spacing between the moment bars reduced the effectiveness of the joists up to their rated capacity. It is believed by the authors that the special anchorage afforded by the welded steel cage, along with the factory controlled conditions of manufacturing the joists, justifies a more thorough investigation with a view toward allowing a closer spacing than that now specified by the ACI code.

EXPLANATION OF TABLE 2

Column

(2) $V = \frac{W}{4}$ W = Total load on 2 joists

(3) $V = \frac{W}{4}$ W = Total load on 2 joists

(4) $V = v_c a_c + v_s a_s = v_c a_c + n_1 v_c a_s$ a_s = area of all longitudinal steel

$v_c = \frac{V}{a_c + n_1 a_s}$ a_c = area of concrete for uncracked section

$n_1 = \frac{E_s\ (s)}{E_c\ (s)} = 7.7$ E_s = modulus of elasticity in shear

(5) $v_c = \frac{VQ}{I_t b}$ (subscript t indicates transformed section) Q for uncracked section; I_t for uncracked section

(6) Same as (4) except a_c = area of concrete above neutral axis

(7) Same as (5) except I_t is for cracked section; Q is for cracked section

(8) $u = \frac{V}{\Sigma o j d}$ o = 50 percent for J1, 45 percent for J2, 40 percent for J3, 35 percent for J4

(9) Same as (8)

(10) $M = V \times 39$ in.

(11) $M = V \times 39$ in.

(12) $f_s/n = \frac{Mc}{I_t}$ I_t for uncracked section

(13) $f_s/n = \frac{M_c}{I_t}$ I_t for cracked section

(14) $f_c = \frac{Mc}{I_t}$ I_t for uncracked section

(15) $f_c = \frac{M_c}{I_t}$ I_t for uncracked section

(16) $f_c = \frac{M_c}{I_t}$ I_t for cracked section

TABLE 2—PRECAST JOIST TESTS

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Joist No.	Shear, V (kips)		v (psi) At 1st crack		v (psi) At failure		Unit Bond u (psi)		Bending moment, in.-kips		Tensile steel f_s (psi)		Tension f_c (psi)	Compression f_c (psi)	
	At 1st crack	At failure	Ave.	Max.	Ave.	Max.	At 1st crack	At failure	At 1st crack	At failure	At 1st crack	At failure	1st crack	At 1st crack	At failure
J1N1	2.46	8.23	68	167	412	520	159	530	96.0	321.0	9700	65200	1135	1135	4490
J1S1	4.03	8.23	112	274	412	520	260	530	157.0	321.0	15900	65200	1860	1860	4490
J2N1	3.00	9.35	83	203	459	590	214	668	117.0	364.0	11850	73800	1380	1380	5090
J2S1	3.50	9.35	97	237	459	590	250	668	136.5	364.0	13800	73800	1615	1615	5090
J2N2	4.10	10.00	113	277	502	632	294	714	160.0	390.0	16200	79100	1890	1890	5450
J2S2	4.10	10.00	113	278		632	294	714	160.0	390.0	16200	79100	1890	1890	5450
J3N1	1.48	8.95	41	100	450	565	190	722	57.7	350.0	5850	71000	682	682	4900
J3S1	3.00	8.95	83	203	450	565	242	722	117.0	350.0	11850	71000	1380	1380	4900
J3N2	2.05	9.75	57	139	490	615	165	786	80.0	381.0	8100	77300	945	945	5330
J3S2	2.05	9.75	57	139	490	615	165	786	80.0	381.0	8100	77300	945	945	5330
J4N1	1.55	8.48	43	105	426	536	143	780	60.5	331.0	6140	67100	715	715	4630
J4S1	1.55	8.48	43	105	426	536	143	780	60.5	331.0	6140	67100	715	715	4630
J4N2	1.00	7.75	28	68	389	490	92	715	39.0	302.0	3950	61200	411	411	4230
J4S2	2.50	7.75	69	170	389	490	230	715	97.5	302.0	9860	61200	1150	1150	4230
J5N1	2.54	11.75	65	181	450	740	150	690	99.0	459.0	6460	65500	943	943	4750
J5S1	3.54	11.75	90	252	450	740	208	690	138.0	459.0	9000	65500	1315	1315	4750
J6N1	3.00	11.20	76	214	430	705	198	740	117.0	437.0	7630	62500	1115	1115	4520
J6S1	3.00	11.20	76	214	430	705	198	740	117.0	437.0	7630	62500	1115	1115	4520

$$n = \frac{E_s}{E_c} = \frac{30 \times 10^6}{3.5 \times 10^6} = 8.57$$

$$n_1 = \frac{G_s}{G_c} = \frac{12 \times 10^6}{1.56 \times 10^6} = 7.7$$

For J1-J2-J3-J4

	Cracked section	Uncracked section
A_c =	9.62	25.88
A_s =	1.34	1.34
$Q = \bar{y}A$ =	30.05	57.35
I =	238.79	423.28
Kd =	3.34	5.00

For J5-J6

Cracked section	Uncracked section
9.26	24.10
2.18	2.18
42.00	74.74
332.90	524.50
3.45	5.00

Experience with factory-made precast concrete members indicates that present Code requirements on bar spacing seem unnecessarily restrictive. The author suggests changes to bring specifications closer to practice.

Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members*

By ARSHAM AMIRIKIAN†

SYNOPSIS

The use of relatively small size aggregates and favorable conditions for quality control make it possible to place reinforcing bars in precast concrete work at closer spacings than those specified or required in conventionally poured-in-place concrete construction. Better quality control and utilization of rich mixtures make it also possible to obtain corrosion protection of reinforcing with relatively thin covers. Recommendations are given for a new basis of specifying bar spacing and cover, in the form of proposed specifications applicable to precast concrete work, together with a discussion of some of the considerations justifying the suggested specifications.

INTRODUCTION

The limitations of minimum bar spacing presently specified in ACI and other building codes when applied to precast concrete elements result in sections much larger than design and construction considerations would normally require. Unlike conventional work, where the use of some excess material in a framing does not appreciably affect its cost, any added weight or section in a precast framing is reflected in the form of an almost proportionate increase in cost. In some projects such additions may not only cause waste of materials but also may render them unprofitable undertakings. Obviously, the new technique of construction can ill-afford to carry such a needless burden during its critical period of development.

It is frankly admitted that in the preparation of the ACI Code no thought was given to the special conditions which now prevail in modern

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precast concrete construction. It is also true that there was no intention on the part of the compilers of the code that it should serve as a guide in all types of work. The new technique differs in many respects from the old. These differences, in turn, call for corresponding changes in specification requirements. New specifications for minimum bar spacing and protective cover constitute the two most important problems in the contemplated task.

SPACING OF BARS

The primary consideration for determining a satisfactory spacing of bars in a member must necessarily be concerned with physical limitations affecting the placement of the concrete mix around the reinforcing. To obtain a good envelope of concrete around each bar, there must be no hampering of the free flow of the aggregates during the placing or casting of an element. In this consideration, the maximum size of the coarse aggregate becomes the dominant factor. To assure its free passage, the gap between two adjacent bars, as well as the clear distance between a bar and the face of the form, must conform to the maximum cross-sectional dimension of the coarse aggregate. That is to say, the provided gap or space must be made sufficiently wide so that the largest aggregate of the mix could pass through freely. Accordingly, this condition will be met if the clear distance is made to equal the maximum cross-sectional dimension of the largest aggregate chosen for the mix plus a practical margin for clearance. For convenience of specification, the margin of clearance may be expressed in terms of the size of the aggregate so that the required total distance can be given in terms of the size of the coarse aggregate alone.

The concept of tying the bar spacing to the size of the aggregate is not new. It is also used in the pertinent clause of the ACI Code, although as a supplementary condition to two other considerations. According to the code, the clear distance between bars may be made as little as "one and one-third times the maximum size of the coarse aggregate," *but* "not less than one inch." Now, in normal concrete construction, where coarse aggregates of a size less than ¾ in. are seldom utilized, these two requirements may be considered as reasonable and consistent. However, in precast concrete construction the so-called coarse aggregate may be as small as ¼ in. Accordingly, if the same specification is used in such cases, the compatibility of the two requirements quoted above will cease, and the requirement for one-inch clear distance will become the sole determining factor. There is no justification for the adoption of such a severe limitation in precast concrete work.

In addition to the abandonment of the one-inch clear distance clause, it is proposed to omit also a third requirement for spacing called for by

the present ACI Code. This concerns the well-known relation between bar dimension and spacing. As in the case of the one-inch clear distance requirement, the background of this clause is vague. The writer's search of technical literature has revealed no experimental or theoretical data to validate the basis of this specification. The basic consideration in this respect is that the enveloping concrete must be adequate, first in bond to transfer the increments of stress from the bars and then, secondly, to transfer the same stresses through the successive layers of concrete by shear. As regards the first, it is believed that the spacing of bars would have no appreciable effect on the developed value of bond, since the critical zone is located at the periphery of the reinforcing. For a given size of rod, the amount of the developed bond will vary with the quality of the surrounding concrete. Owing to the more favorable conditions of placement and quality control, it is reasonable to expect that the bond developed in precast work would be greater than that obtained in conventional concreting. When considered for shear, the critical zone becomes the narrowest gap between the bars. While it may be possible to specify this gap in terms of bar diameter or dimension, a spacing thus obtained must then be made to vary in accordance with the arrangement of the reinforcing. To elaborate on this, consider the case of a beam having two bottom layers of reinforcing, consisting of an equal number of bars of the same size in each layer. According to specification requirements, the rods may be spaced the same distance apart in both layers. However, if this is done, the unit shear in the gaps of the second layer will be approximately three times that at the corresponding locations in the first layer. Furthermore, it is to be noted that the specified clear distance between the bars does not provide a balance between bond and shear. The unit value for bond given in the code is $0.05f'_c$, and that for shear in concrete is $0.02f'_c$. With these values, a balanced arrangement would require, for the full development of bond, a clear distance equal to 3.9 bar diameters, in lieu of the 1.5 diameter prescribed by the code. In view of those considerations, the omission of this requirement in the proposed specification for precast concrete work is not of serious importance.

PROTECTIVE COVER

Contrary to prevailing concept, the thickness of concrete alone is not a true measure of protection against corrosion. The quality of concrete surrounding the reinforcing is of much more importance than its thickness. A very thin layer of rich concrete (containing, say, 7 to 8 sacks of cement per cu yd) properly graded, placed under controlled conditions and vibrated, would provide a cover impervious to moisture. Navy experience with precast concrete structures indicates that the thick-

ness of protective cover could be reduced to as little as ¼ in. This is corroborated by a research program now in progress at Virginia Polytechnic Institute.*

As regards the consideration for fire protection, it is admitted that the reduced thickness would be at the expense of decreased rating, but not necessarily in a direct ratio. A projected laboratory investigation should furnish the needed information in this respect.

CONCLUSION

The foregoing discussion is presented as background material for the justification of the following proposed specifications, intended to be incorporated in a code for precast concrete construction.

1. Placing of reinforcement

(a) Reinforcing bars or mesh shall be accurately placed and adequately secured in position by concrete or metal chairs and spacers. Where the reinforcement comprises wire mesh and reinforcing bars, or reinforcing bars and bands or stirrups, preassembly of such reinforcement in cages, or trusses, by welding in a template or jig before placing in the forms is recommended.

(b) The minimum clear distance between parallel bars shall be one and one-half times the maximum size of the coarse aggregate.

2. Protective cover for reinforcement

(a) For surfaces exposed to weather, ground or in contact with water: Main reinforcing in beams, girders and columns shall be protected with concrete equal in thickness to two times the maximum size of the coarse aggregate, but in no case shall the thickness of covering be less than ½ in. Reinforcing of slabs and secondary reinforcing in beams, girders and columns shall be protected with concrete equal in thickness to one and one-half times the maximum size of the coarse aggregate, but in no case shall the thickness of covering be less than ⅜ in.

(b) For surfaces not exposed to weather: Main reinforcing in beams, girders and columns shall be protected with concrete equal in thickness to one and one-half times the maximum size of the coarse aggregate, but in no case shall the thickness of covering be less than ⅜ in. Reinforcing of slabs and secondary reinforcing in beams, girders and columns shall be protected with concrete equal in thickness to one and one-quarter times the maximum size of the coarse aggregate, but in no case shall the thickness of covering be less than ¼ in.

*Pletta, D. H., Massie, E. F., and Robins, H. S., "Corrosion Protection of Thin Precast Concrete Sections," ACI JOURNAL, Mar. 1950, *Proc.* V. 46, pp. 513-528.

Discussion of a paper by Arsham Amirikian:

Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members*

By S. J. CHAMBERLIN, L. J. MENSCH and AUTHOR

By S. J. CHAMBERLIN†

The writer is not greatly concerned about any appreciable reduction in bond with closer spacing of the reinforcing steel; it is possible to get the proper bond in precast work with small aggregates, good placement methods and high quality concrete carefully controlled. But the writer is worried about the net concrete area between the bars required to carry the load developed by the bond on the under side of the bars up into the beam. It has to be carried by shear. Some tests at Iowa State College indicate that the more horizontal shear area there is right at the plane of the centerlines of the bars (wider spacing) the higher the loads possible on beams before failure. Is it not a question of carrying the load developed on the under side of the bars through shear up into the beam?

The writer believes that the precast joists described would hold up without any concrete below the neutral surface. There is a steel truss in there; it is different than a beam.

The writer would like to point out that there must be sufficient shear area to bring the load developed on the underside of the bars up into the beam.

By L. J. MENSCH‡

The writer agrees with the author's suggestions but does not agree with the restrictions of the Joint Committee Report.

When the writer started in the concrete business over 50 years ago, it was the practice to place not one layer of steel reinforcing, in girders, but generally two layers and often three layers. In fact, we even built four

*ACI JOURNAL, Apr. 1950, *Proc.* V. 46, p. 637. Disc. 46-44 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

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layers—with 2-in. bars, not ½-in. or ⅝-in. bars.

We built some girders in Cincinnati in 1902 with a 61-ft span; they were 12 in. wide, 48 in. deep and had 8 bars of 2-in. diameter. Now, that is only a 12 in. wide beam and it had two layers of 2-in. bars. These girders have been standing for 48 years.

T. D. Mylrea published beam tests 15 years ago* which showed the whole beam filled with steel rods so that there was hardly any concrete between the steel rods and the stresses ran over 70,000 psi.

So just where do code makers get their requirements?

AUTHOR'S CLOSURE

The writer does not share Chamberlin's concern regarding the effect of reduced bar spacing on the load carrying capacity of a reinforced concrete beam. Theoretically this should be an important factor in the proportioning of a section subjected to high shear. However, since shear is a function of increment of stress, the requirement for so-called horizontal shear area would seem to be a much more serious problem in multiple-layer arrangements of standard specification bar spacing than in a single-layer arrangement of reduced spacing. Now, our experience with the former arrangements, as vividly described by Mensch, does not reveal any apparent weaknesses traceable to this cause. This, in turn, may be taken as indicative of a faulty concept on our part of either shear capacity or true flexural behavior of a reinforced section.

In the absence of details of the tests mentioned by Chamberlin, it is difficult to comment on the observed results. The higher load values obtained with some specimens may have been due to factors or conditions other than wide bar spacing. Also, higher flexural capacity may not necessarily signify a correspondingly higher shear strength.

If the reinforcement is arranged to form a truss, either by welding the assembly or by effectively tying the webbing to top and bottom rods, unquestionably the shear will be taken by truss action rather than by the concrete. Tests of precast concrete joists corroborate this concept.† Since this is the condition anticipated in the proposed specification, then the reduced bar spacing should not be considered as a factor which might reduce the shear capacity of a member.

*Mylrea, T. D., "Tests of Reinforced Concrete T-Beams," ACI JOURNAL, May-June 1934, *Proc.* V. 30, pp. 448-464.

†Menefee, F. N. and Kinnier, H. L., "Spacing of Moment Bars in Precast Joists," ACI JOURNAL, Apr. 1950, *Proc.* V. 46, pp. 629-636.

A statement of premises from which a recommended practice for the design of concrete pavements may be developed.

The Problem of Slab Dimensions*

Reported by Subcommittee 2, ACI Committee 325

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SYNOPSIS

The dimensions of concrete pavement slabs necessary for optimum results depend on and are influenced by a great many factors, economic as well as physical. The factors and their relationship to slab dimensions discussed include strength, elastic and plastic properties, volume change, subgrades, construction conditions and slab thickness, length and width.

The subject assigned to Subcommittee 2, Committee 325, has the seemingly innocent title of "Slab Dimensions." This sounds like a simple topic, but it is in reality a complicated one, for the dimensions of concrete pavement slabs necessary for optimum results depend on and are influenced by a great many factors, economic as well as physical.

No conclusions regarding slab dimensions have been reached, and at present the subcommittee will merely appraise the problem and state what it involves. Later it hopes to examine critically the information available, and if needed facts are missing, it will attempt to bring them to light through the stimulation of research.

One familiar with the general problem of concrete pavement design could readily set down in systematic order a tabulation of the many factors upon which the dimensions of pavement slabs, their length, width and thickness, must depend. It is our present purpose to consider these factors very briefly and discuss their relationship to our problem. It would be premature to even attempt to set up a design procedure, but rather it seems desirable to stand off and view the problem

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in its many aspects. It is hoped that the development of a logical procedure for pavement design will be the ultimate result; at least, that is our goal.

Now, what are these various factors which have influence on slab dimensions? Let us consider those which are most important, and in just enough detail to see how they bear on our problem.

SIGNIFICANT PROPERTIES OF PAVEMENT CONCRETE

Strength

Obviously, the strength of concrete is of primary significance. When a crack occurs either there is excessive shearing stress or, more likely, the tensile strength of the concrete has been exceeded. Excessive tension may be produced by direct external force, by internal force, or by flexure. Expansion of the concrete from whatever cause may produce direct compression which may be accompanied by "blow ups" in the pavement and high shearing stress near the position of failure.

Concrete requires considerable time in which to attain full strength, its strength during the first few days of its life being much lower than its ultimate strength. In addition, as compared with its compressive strength, the tensile strength of concrete is relatively very low. Whereas the 28-day compressive strength of pavement concrete usually ranges from 4000 to 6000 psi, its tensile strength rarely if ever exceeds 600 psi —from 300 to 500 psi being typical.

A concrete slab is essentially a beam subjected to bending stresses, and so beam or bending resistance as measured by modulus of rupture has become recognized as a most significant property of pavement concrete. But modulus of rupture, like compressive strength, depends upon the method of loading and on the shape and other characteristics of the specimen. Third-point loading gives results some 10 to 15 percent lower than single-point, center loading, and the question naturally arises as to what the standard third-point loading test means in terms of the bending stress produced in a road slab under actual loading conditions.

It is to be remembered, also, that a road slab, although subjected to static stress when vehicles are at rest or when the forces due to moisture and temperature are acting, for the most part is subjected to dynamic loads applied repeatedly. The stress-producing effects of dynamic loads and the resistance of concrete to fatigue must be properly evaluated and properly used in pavement slab design theory.

Elastic and plastic properties

The elastic properties of concrete, including such matters as the stress-deformation relationship from which Young's modulus is determined, must be taken into consideration in any design theory as must

also Poisson's ratio, which is a measure of lateral deformation accompanying axial stress. The higher the modulus of elasticity the greater is the stress for a given deformation in the concrete and the less yielding and more rigid is the slab, facts which already have stimulated the imagination of some advanced thinkers in slab design. Of importance equal to that of elastic properties is the study of the plastic properties, for concrete flows or deforms under sustained load, such as might result from moisture or temperature expansion. Obviously, this plastic property of concrete must have a beneficial effect in decreasing the frequency of "blow ups" and an effect also on slab length. But it must likewise influence allowable unit stress, for will not concrete crack if deformed or stretched excessively even though the accompanying stress is low?

Volume change

Concrete is altered in volume by changes in both temperature and moisture. For the most part, moisture volume changes are due not so much to the aggregates as to the cement, and hence, to reduce moisture volume change in concrete the cement factor should be reduced to a value compatible with other controlling essentials, such as strength. The aggregates occupy a large percentage of the volume of concrete, and so they, and especially the coarse aggregate, will control the volume changes due to temperature. Aggregates vary greatly in thermal expansion properties and aggregates commonly used have thermal coefficients of expansion ranging from 0.000003 to 0.000007. Concretes made from these aggregates will have correspondingly high volume change differences.

Obviously, volume change produces direct tension or compression in a pavement slab and, likewise, because of differential change in either temperature or moisture from the top to the bottom of the slab, warping stresses sometimes equal to load stresses may be produced. The importance of maintaining volume change at a minimum is at once realized. The omission of aggregates having extremely high or extremely low thermal coefficients, the proportioning of concrete for use of a low cement factor, dry consistency, and other expedients may prove helpful.

Concrete proportioning may affect moisture shrinkage because of differences in mixing water and amount of cement. Different cements expand differently, and chemical composition in the past has played an important part in the volume changes in concrete.

Aggregate characteristics are likewise important, for it has been found that certain aggregates, although sound in standard laboratory tests, will expand excessively due to freezing and thawing. Wetting and drying, alternations of temperature, and chemical reactivity between the cement and aggregates may also have an expanding effect

on the concrete. Pavements built with such aggregates have shown excessive expansion as evidenced by closing of joints and "blow-ups" and later by the formation of "D" cracking—that is, frequently spaced cracks in all directions, all generally filled with a hard deposit from which such cracks have derived their designation. It is doubtful if design should attempt to allow for defective materials, but economics may make that desirable.

EFFECT OF SUBGRADE

The subgrade plays a very important part in the design and in the service life of a concrete pavement, and too little attention has been paid to this fact in the past. A number of questions arise in connection with the subgrade. What should be its characteristics as far as bearing value, permeability, drainage, resistance to mud pumping, capillarity, swelling and shrinkage, elastic properties, and effect on subgrade "friction" are concerned? Many concrete pavements have been laid with no attention whatever to subgrade characteristics, and an evaluation of the above mentioned properties and a determination of the most desirable characteristics of the subgrade upon which a concrete pavement is to be built is necessary. If a poor subgrade must be used, how thick should a base course layer be to overcome the ill effects of the poor subgrade? Is it feasible to reduce the thickness of a concrete slab because of the use of a good subgrade material as compared with a poor subgrade, or with a given thickness of base course as compared with no base course? Some measure of the bearing resistance of the subgrade enters into most design theories. The swelling and shrinkage characteristics of the subgrade will partially determine the permanency of good riding qualities. Obviously, if the subgrade material swells near the shoulders and near leaky expansion joints in the wet season and shrinks in the dry season, the slab receives extremely variable subgrade support and an originally smooth riding pavement soon becomes rough. The resistance to mud pumping will likewise determine the ability of the slab to maintain its surface in a smooth riding condition and, likewise, to support heavy wheel loads. Capillarity in the subgrade controls the rate at which water is fed up through the concrete slab from the underlying subgrade, and hence, capillarity is important since it may affect warping, extent of swelling due to moisture, scaling, and other phenomena.

Subcommittee 1 has been set up especially to study the subgrade and its effects on the concrete pavement and obviously the work of that subcommittee* must be taken into account for it has a most important bearing on slab dimensions.

*Woods, Kenneth B., "Influence of Subgrades and Bases on Design of Rigid Pavements," ACI JOURNAL, Jan. 1950, *Proc.* V. 46, pp. 329-348.

CONSTRUCTION CONDITIONS

Construction conditions, including the method of placing, consolidation, finishing and curing, are important because they affect properties of the concrete which influence cracking. For illustration, the method of finishing the concrete, whether by vibration or otherwise, will affect the strength, consistency, concrete proportions and shrinkage. The placing of concrete under some temperature conditions, for example, a great drop in night temperature, will produce high contraction at a time when the concrete is weak and the result will be transverse cracking during the first 24 hours after placing.

The moisture condition of the subgrade at the time of placing the concrete becomes important because of the effect of an absorbent subgrade in the excessive extraction of moisture from the fresh concrete. The proper degree of compaction of the subgrade and the right amount of moisture are necessary for optimum stability. Unequal swelling of the subgrade due to absorption of water from the concrete after it is placed results in an uneven slab surface. The subgrade is an important part of the pavement as a whole and its proper construction must not be neglected.

Effective curing protection of the slab against rapid temperature change tends to prevent transverse cracking at an early period. The effectiveness of curing methods also controls the properties of the hardened concrete, and hence, curing becomes an important construction item. It definitely affects the length of pavement slabs.

THICKNESS OF SLAB

It goes without saying that thickness is a most important slab dimension, for the very life of the pavement depends on the thickness being adequate to support the heaviest allowable wheel loads. Many problems arise in considering proper thickness design.

What is the effect of wheel loads and wheel load frequency? An answer to this question requires a knowledge of the effect of repeated load application on the fatigue resistance of concrete and the strength recovery during rest periods.

Design theory, although in general found to be excellent, needs re-examination, particularly with regard to certain assumptions concerned primarily with subgrade support. A road slab does not remain in uniform contact with the subgrade as required by the most widely accepted theory, and this lack of uniformity in subgrade support needs re-evaluation. Passing loads distort the subgrade and temperature and moisture differentials warp the slab, and thus, only by the rarest chance can there be uniformity in pressure between the subgrade and the slab.

Speed of load applications must affect the degree of subgrade support, the "k" value in the Westergaard theory and its effect need evaluation.

Tires, especially those on large planes, cover large areas of contact and the effect of these contact areas as they span across joints needs thorough inquiry. If a tire contact area is 50 in. long and 30 in. wide, it is difficult to imagine a true corner loading; a single corner could hardly carry the entire wheel load. Similarly, such a wide area of tire contact renders it difficult to place an entire wheel load at the extreme edge of the slab; hence, something less than a wheel load is supported by a corner or an edge of a runway slab. No doubt other matters in the design theory of a highly specialized nature should be re-examined.

What is the influence of width on edge thickness? There is a trend away from thickened edge design as the width of pavement lanes has been increasing, due possibly to the decreased frequency with which heavy wheel loads travel over the corners of slabs when the pavement is wide enough to accommodate traffic. This brings up the matter of the proper shape of cross section.

What is the role of reinforcing steel and what is its effect on the required thickness of slab and on the control of pavement cracking? This is a separate study to be undertaken by a special subcommittee.

LENGTH OF SLAB

One of the most controversial subjects today in concrete pavement design is the proper spacing of transverse joints. Opinions vary all the way from no joints at all to frequently placed contraction joints, as little as 15 ft apart. The expansion and contraction of concrete due to temperature change and the swelling and shrinkage due to wetting and drying are well-known phenomena. Concrete swells when it absorbs moisture and shrinks as it dries, but these changes are less than the expansion and contraction in a pavement resulting from temperature change.

Studies are needed on the influence of slab length on direct tension and direct compression; also the influence of slab length on warping stresses due to temperature differential from top to bottom of the slab.

Is it best to install joints or should the pavement be allowed to crack as it will with no control whatever? It is certain that cracks will form if joints of one kind or another are omitted. It is recognized that all joints as now built are points of potential weakness; certainly they are points of nonuniformity in the elastic properties of the slab. Are expansion joints necessary, and if so, how frequently should they be placed? What governs their frequency? The same inquiry might be made with regard to contraction joints and warping joints. What are the reasons

for and against a continuous pavement with no joints whatever? If joints can be eliminated, under what circumstances can such construction be used successfully and what precautions must be taken? What is the relationship between amount of longitudinal steel and slab length? The committee recognizes that many arguments have been advanced for and against expansion joints and contraction joints, but it is far from ready to express itself on this most important matter.

SLAB WIDTH

Slab width should be a relatively easy part of pavement design to settle rather conclusively. In general, concrete pavements are called upon to carry the heaviest and largest units of traffic at relatively high speeds and the width must be such as to permit these units to operate safely. In the October, November, and December, 1945, issues of *Public Roads*, a research report is published entitled "Effects of Roadway Width on Vehicle Operation." As a result of this information, the Bureau of Public Roads and the American Assn. of State Highway Officials adopted "A Policy Concerning Maximum Dimensions, Weights, and Speeds of Motor Vehicles to be Operated Over the Highways of the United States." The standard of width recommended in this "Policy" is: "No vehicle, unloaded or with load shall have a total outside width in excess of 96 inches." In addition, a special note urges that consideration be given to the desirability of providing for vehicles 102 in. wide in planning the reconstruction of Federal Aid and State Highways.

The basic unit of width is the traffic lane and the conclusions reached by research and experience seem to point to the desirability of using a 12-ft lane. This width is considered as minimum when and if vehicles 102 in. wide come into general use. However, many other considerations regarding width will confront the committee, such as questions of traffic capacity, widening on curves, and the possibility of longitudinal cracking.

RECOMMENDED PRACTICES

The ultimate aim of the work of the committee will be to devise a set of recommended practices covering concrete pavement design. Some agreement is needed regarding acceptable theory for determining pavement thickness which will realistically take into account the physical phenomena concerning concrete pavements as revealed by research. In general, test data take on real significance when they can be generalized into a workable theory and similarly the various researches which the committee must study will become useful to the extent that they may be utilized in the formulation of some recommended practices for concrete pavement design.

CONCLUSION

It will be apparent that the committee is not jumping at any conclusions; too much of that has been done in years gone by. The present report pretends to do nothing more than state something of the problem and give an idea of the many separate items which must be considered before any final report can be completed. An outline of work topics has been prepared and it is the hope of the committee that ways will be found to expedite to the greatest possible extent a study of these topics to the end that design recommendations may be forthcoming in the not too distant future.

Given—Air entrainment
Known—Effect on compressive and flexural strength
Wanted—Effect on another important property of reinforced concrete structures—

Effect of Entrained Air on Bond Between Concrete and Reinforcing Steel*

By EIVIND HOGNESTAD† and C. P. SIESS‡

SYNOPSIS

This paper reports an experimental investigation of the effect of entrained air on the bond properties of the new Laclede and the Hi-Bond bars, each in ½ in. and ⅞ in. sizes. Bond properties were studied through 108 pull-out tests with a constant bar embedment of 12 diameters. Bars were cast vertically as well as horizontally; in the latter case, with 6 and 30 bar diameters depth of concrete under the bars. A 5000 psi basic concrete mix was used, with air content as a major variable.

Within the scope of these tests it was found that up to the normal air content of 4 to 5 percent, the bond of all bars in all positions was reduced, in terms of percent, less than the flexural and compressive strength of the concrete. Where more than 5 percent air was entrained, however, the bond of the horizontal bars was reduced rapidly, probably due to rising air.

INTRODUCTION

Air entrainment in concrete has been credited with being a major development in the field of concrete engineering, comparable in importance to the discovery of the water-cement law. The use of air entrainment has been subject to intense laboratory studies, and considerable practical experience has been gained primarily with respect to highway and airfield pavements.

During the last few years air entrainment has entered the field of bridge and structural engineering. This introduces no new problems as far as compressive and flexural strength are concerned since the effects of air entrainment on those properties have been rather well established. The bond between concrete and steel is, however, another

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property of considerable importance in reinforced concrete structures. Relatively little is known about the effects of air entrainment on this property of steel and concrete. The only reported tests on this effect known to the writers are those carried out by the Corps of Engineers.[1,2]

The purpose of the tests reported herein is limited to evaluating the effects of air entrainment on bond properties of concrete having a cylinder strength of 4000 to 5000 psi, which corresponds to the concrete generally used for highway structures in Illinois. Nevertheless, it is believed that the results obtained are worthy of a wider circle of interest.

Outline of tests

It was recognized in planning these tests that all of the many variables which affect the bond between concrete and steel could not be investigated without an extensive program. Consequently, it was decided to restrict the number of variables to a minimum consistent with the purpose of the tests.

All tests were of the pull-out type in which the bars were pulled from a concrete cube having a side length of 12 times the bar diameter. A total of 108 specimens were tested at an age of 28 days. Slip between the bar and the concrete cube was measured both at the loaded and the free end of the bar.

The variables considered included three nominal air contents, two types and two sizes of bars, and three positions of casting: vertical, and horizontal top and bottom. Three companion specimens embodying each combination of variables were made.

MATERIALS, FABRICATION AND TEST METHOD

Materials

Reinforcing steel—Since the main object of these tests was to determine the effect of air entrainment on bond, and not to compare different commercial types of bars, only the new (1948) Laclede and the Hi-Bond bars were used, both in ½-in. and ⅞-in. sizes (Fig. 1). The Laclede bar used was representative of bars barely meeting the requirements of A.S.T.M. A 305-47T, "Tentative Minimum Requirements for Deforma-

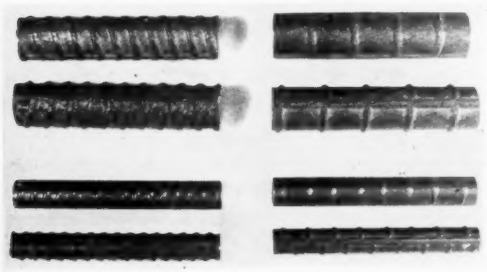

Fig. 1—Types of bars tested, (left) Hi-Bond, (right) Laclede

tions of Deformed Steel Bars for Concrete Reinforcement." The Hi-Bond bar used exceeded this specification by a considerable margin.

The results of tension tests made upon 2-ft test specimens cut from each of the 20 ft lengths in which the bars were purchased are presented in Table 1. It appears that the Laclede bars met the requirements of A.S.T.M. Designation A 16-35 for rail steel reinforcement, while the Hi-Bond bars met those of A.S.T.M. Designation A 15-39 for intermediate grade billet steel, all by a considerable margin.

A careful study of the bar deformations was made to obtain information concerning the relationship between lugs and bond. In determining the characteristics of the lugs, the bar to be measured was placed in a lathe, and two Ames dials reading to 0.001 in. were mounted so as to furnish a Cartesian coordinate system. One dial, equipped with a pointed tip bearing against the specimen, was mounted in the tool holder with the stem of the dial perpendicular to the axis of the specimen; the second dial measured the longitudinal movement of the carriage. By this means, diagrams similar to those shown in Fig. 2 were obtained. The sections shown in these figures are parallel to the axis of the bar. Therefore, the sections for the Hi-Bond bars are not perpendicular to the lugs.

The height of lugs was determined from the diagrams. The average height at the center of the lug for each type and size never differed more than 10 percent from the height measured at the quarter points; it was not considered necessary, therefore, to use a weighted average in favor of the measurements at the quarter points as indicated in A.S.T.M.

TABLE 1—TENSION TESTS OF REINFORCING BARS

Type of bar	Size in.	Average* stress at yield point, psi	Max. and min. yield point, psi	Average ultimate strength psi	Average elongation in 8 in. at ult. load, percent	Average reduction in area† at ult. load, percent	Average modulus of elasticity, psi × 10^6
Laclede	⅞	60,430	66,000 59,900	108,200	15.1	29.3	28.6
	½	64,100	68,400 59,500	123,100	23.3	24.0	28.3
Hi-Bond	⅞	50,440	54,010 47,050	88,100	20.0	32.0	29.0
	½	44,250	45,500 42,500	73,400	22.7	44.0	28.6

All stresses, including the modulus of elasticity, are based on the nominal cross-sectional area: 0.20 sq in. and 0.60 sq in. for ½-in. and ⅞-in. bars, respectively.

*Average of 5 specimens for the ⅞-in. bars, 4 for the ½-in. bars.

†The diameter from gap to gap (d_1 in Fig. 3) was measured before testing and after failure. The minimum diameter, d_2, perpendicular to d_1 was also measured twice. The reduction in area is computed as the change in $(d_1 + d_2)^2$.

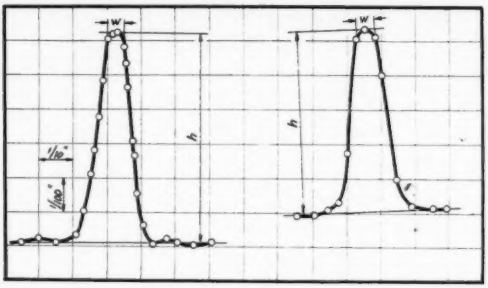

Fig. 2—Characteristic lug profiles for ⅞-in. bars, (left) Laclede, (right) Hi-Bond

A 305-47T, 4(b). Four measurements were made on each specimen as indicated in Fig. 3. An average of all measurements of lug height, h, is shown in Table 2. The spacing of the lugs, s, was determined by measuring across 10 lugs for the ⅞-in. bars and 20 lugs for the ½-in. bars. Two measurements were made on each side of each specimen.

The gap, g, and outer diameters, d_1, d_2, (Fig. 3) were measured with a micrometer. In no case was the difference between d_1 and d_2 at the same section more than 5 percent. Therefore, a simple average, d, (Table 2) could be used. The bearing area was computed as:

$$\text{Bearing area} = [\pi(d - h) - 2g]\, h$$

This area was expressed in square inches per inch by dividing by the lug spacing, s. The bearing area in terms of square inches per square inch nominal surface area is also shown in Table 2.

The shearing area of a commercial bar is generally not clearly defined. The top of the lugs often are well rounded as may be seen in Fig. 2. Thus, there is some basis for defining the shearing area as a cylinder touching the top of the lugs with a deduction for the gap only. An inspection of a Hi-Bond bar after shearing failure, however, disclosed that the concrete around the top of the lugs was not effective in resisting shear[3]. Therefore a small area, in addition to the gap, should be deducted to comply with this observation. To accomplish that, the sides of the lugs were projected until they met the cylinder touching the top

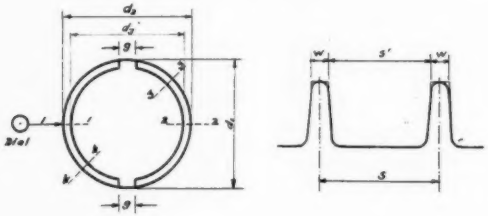

Fig. 3—Lug measurements

TABLE 2—LUG MEASUREMENTS

Bar diam., in.	Type of bar	Average actual weight, lb per ft	Dimensions of bars				Requirements of A.S.T.M. Designation A 305-47T				Bearing area		Shearing area, sq in. per in.
			Height* (h), in.	Spacing* (s), in.	Gap* (g), in.	Diam.† (d), in.	Min. height		Maximum spacing, in.	Max. gap 12.5 percent of nominal perimeter	Sq in. per in.	Sq in. per sq in. nom. surf. area	
							6.5percent of actual spacing	Abs. min.					
⅞	Laclede	2.037	0.055	0.861	0.136	0.955	0.056	0.044	0.875	0.344	0.164	0.060	2.56
⅞	Hi-Bond	1.998	0.048	0.380	0.130	0.923	0.025	0.044	0.875	0.344	0.314	0.114	2.13
½	Laclede	0.675	0.028	0.502	0.099	0.547	0.032	0.020	0.500	0.191	0.080	0.051	1.37
½	Hi-Bond	0.672	0.036	0.253	0.075	0.547	0.016	0.020	0.500	0.191	0.206	0.131	1.22

*Average of 20 measurements for ⅞-in., 16 for ½-in. bars.
†Average of 40 measurements for ⅞-in., 32 for ½-in. bars.

of the lugs, and the shearing area, recorded in Table 2, was defined as follows:

$$\text{Shearing area} = (\pi d - 2g)\frac{s'}{s} \text{ (sq in. per in. length)}$$

The distance w (Fig. 3) was determined as the average of two characteristic lug sections at the center, and two at the quarter points for each type and size bar.

The inclinaton of the bearing faces of the lug projections may be of some significance. This angle, as shown magnified in Fig. 2, is of the same order of magnitude (40 to 50 degrees) for all four bars included in these tests. The inclination of the lugs relative to the axis of the bar is 90 degrees for the Laclede bars and about 70 degrees for the Hi-Bond bars. This difference of 20 degrees is not believed to be of any importance as far as bond is concerned.

Aggregates and cement—The fine aggregate was a Wabash River torpedo sand having an average fineness modulus of about 2.9. The coarse aggregate was a Wabash River gravel of 1-in. maximum size. Both are aggregates that have been in general use at this laboratory for years and passed the usual specification tests. Standard portland cement purchased from a local dealer and stored under proper conditions was used throughout the tests.

Concrete mixtures—Three main concrete mixtures were used: (1) a basic mix without air entrainment; (2) a mixture containing about 4 percent of air; and (3) a mixture containing about 6 percent, or more, air. The basic mixture was comparable to that used in structural work by the Illinois Division of Highways and had an average 28-day compressive strength of about 5100 psi. The air-entraining mixtures were obtained by modifying the water content and sand percentage of the basic mix, keeping the cement content and slump constant. Duplicate air content determinations were made on each batch of concrete using a pressure-type air meter. Properties of the concrete mixtures are given in Table 3.

The air-entraining agent used was Darex AEA. Eight 6 x 12-in. cylinders and one 4 x 4 x 24-in. beam were made from each batch as auxiliary specimens. Each beam was broken in flexure three times.

Fabrication of test specimens

The types of specimen used in these tests are shown in Fig. 4. Since the effect of the settlement under a horizontally cast bar has an important influence on bond properties, three positions of casting were used: vertical, and horizontal top and bottom.

In the vertical position, the bars projected upward from the cube so that the direction of settlement of the concrete was opposite to the

TABLE 3—PROPERTIES OF CONCRETE

Mix No.	Type of bar	W/C (wt.)	C/(W +L) (volume)	Slump, in.	Net percent air (volume)	Cement, lb per cu yd	Modulus of rupture, psi, 28 days	Compressive strength, psi, 6 x 12-in. cylinders	
								28-day	14-day
A1a	H	0.500	0.612	3.5	1.05	600	950	5130	4510
A1b	L	0.500	0.618	4.0	0.90	602	840	5145	4420
A2a	L	0.495	0.610	3.0	1.30	601	970	5505	4620
A2b	H	0.495	0.623	3.5	0.95	602	830	4835	3850
A3a	H	0.500	0.603	3.0	1.30	599	920	4940	4450
A3b	L	0.500	0.613	4.25	0.95	600	920	4960	4250
A. Ave.		0.500	0.613	3.5	1.08	601	905	5086	4350
B1a	H	0.475	0.554	3.5	3.95	602	900	4810	3910
B1b	L	0.475	0.545	3.5	4.20	601	740	4720	4050
B2a	L	0.475	0.505	4.25	5.85*	593	730	3880	3170
B2b	H	0.475	0.544	4.25	4.30	602	710	4420	3830
B3a	H	0.485	0.510	4.25	5.20	594	760	4270	3770
B3b	L	0.480	0.524	4.0	4.90	597	810	4520	3870
B. Ave.		0.475	0.530	4.0	4.73	598	775	4437	3758
C1a	H	0.445	0.544	3.0	5.40	615	780	4290	3900
C1b	L	0.445	0.523	3.0	6.24	609	720	4190	3730
C2a	L	0.470	0.494	3.5	6.40	604	720	4140	3750
C2b	H	0.470	0.494	3.5	6.40	604	730	4110	3650
C3a	H	0.480	0.478	4.0	7.00	598	770	3790	3380
C3b	L	0.480	0.463	4.25	7.70	594	650	3430	3020
C. Ave.		0.465	0.499	3.5	6.53	604	728	3992	3572

H = Hi-Bond bars
L = Laclede bars
C/(W + L) = Cement-space ratio

*Accidentally about 6 min. mixing time.

Proportions by weight:
Mix A—1 : 1.97 : 3.27
Mix B—1 : 1.78 : 3.27
Mix C—1 : 1.63 : 3.27

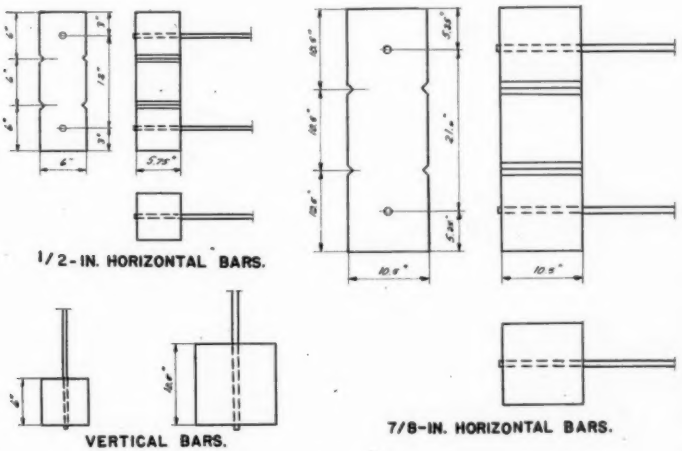

Fig. 4—Sketches of pull-out specimens

direction in which the bar was pulled during the pull-out test. This type of specimen was used to evaluate the effects of entrained air on the bond properties under conditions for which the effects of settlement would be negligible.

Specimens for the horizontally cast bars were prisms 12 bar diameters square and 36 diameters tall. These specimens may be visualized as three cubes stacked on top of each other, with the reinforcing bars extending horizontally through the centers of the top and bottom cubes, the longitudinal ribs being in a horizontal plane. Each specimen was grooved horizontally with V-shaped grooves, and later broken by flexure into three cubes before the pull-out tests were made. The grooves were obtained by attaching small angles to the inside walls of the forms. In breaking specimens into cubes nearly plane fractures were obtained, the surfaces departing from a plane only to about the extent of the maximum size of coarse aggregate. The center cube, which contained no bar, was discarded. This method of casting was used to study the effect of settlement of different depths of concrete under the bars. The depth under the top bar was 30 bar diameters, that under the bottom bar, 6 diameters.

All specimens were cast in watertight steel forms which were removed about 20 hours after casting. However, the forms were carefully designed so as to allow their removal with negligible disturbance of the bars,* and great care was taken not to disturb the bars in handling the specimens during their early age. All specimens were cured in a fog-room and tested damp at an age of 28 days.

Test method

As shown in Fig. 5, the specimens were supported during testing on the fixed head of a universal testing machine by means of a spherical

*Details of the forms were similar to those illustrated in A.S.T.M. C 234-49T, Fig. 1.

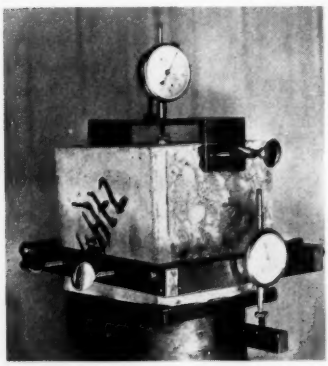

Fig. 5—Pull-out specimen in testing machine

block, a cylindrical block with a horizontal slot, and a square base plate with a linoleum cushion. The reinforcing bar was carried through this rig to the movable grips of the machine.

Slip at the free end of the bar was read directly by means of an Ames dial mounted on a yoke clamped to the concrete cube, with the stem of the dial resting on the planed upper end of the reinforcing bar. Slip at the loaded end, however, was measured indirectly by two dials fastened to opposite sides of a second yoke which was also clamped firmly to the concrete cube. Each dial stem bore against the smooth surface of a bolt fastened to a horizontal eye-bar, which in turn was held firmly around the pull-out bar by three set-screws. As load was applied, the average of the dial readings indicated the movement of the reinforcing bar at the point of attachment of the horizontal eye-bar, with respect to the points on the concrete cube at which the yoke holding the dial gages was attached. In determining the slip at the face of the concrete (slip at loaded end), this average reading was corrected for the elongation of the reinforcing bar between the eye-bar and the concrete surface. The compression of the concrete between the face of the cube and the points of attachment was considered negligible.

Loads were applied to the reinforcing bar through the movable head of the testing machine, generally in steps of 1000 lb for the ½-in. bars, and 2000 lb for the ⅞-in. bars. Loading was continued until bond failure or yielding of the bars took place. In some few cases the bar was pulled beyond the yield point. Violent and dangerous splitting failures took place in these cases, and such testing was therefore discontinued. All three dial gages were graduated to 0.001 in., and were read to 0.0001 in.

DISCUSSION AND ANALYSIS OF TESTS

Selection of criteria

The question of whether or not the conditions in bond pull-out tests are comparable to those in structures is a question which has received considerable attention. The problem is a complicated and difficult one, and much material has been presented pro and con without any definite agreement. In the present tests, the trends of the influence of air entrainment on carefully planned pull-out tests are believed to give helpful information in judging the effects likely to be produced in structures. When considering quantitative information, however, it is believed to be of fundamental importance to know how the pull-out tests are made and interpreted.

Even for a given bar and a particular quality of concrete, the characteristics of the bond between the two, as determined from a pull-out test, depend on several variables, one of the most important of which

is the position of the bar during casting. The settlement conditions under the bar are also significant.

Another important variable related to the test specimen is the length of embedment. Abrams[4] attempted to eliminate its effect by extrapolating to zero embedment. Since then, much information has been obtained about this variable, but the matter is still far from clear. Lately, during the development of the modern deformed bars, most investigators have used two or three different lengths of embedment, at times averaging the results. In the present tests, only one length of embedment was used in order to limit the testing program.

When bond is studied in terms of average bond stress, it is of no importance whether bond performance is presented as steel stress, total load on the bar, or computed average bond stress; there exists a linear correlation between all three quantities. In these tests, nominal average bond stresses were computed from the load, the length of embedment, and the nominal circumference of the bars.

In previous work, data obtained in pull-out tests have been studied on several different bases: (1) bond- or load-slip curves, (2) average bond, or steel stress at a certain slip, and (3) maximum average bond, or load at splitting of the concrete specimen.

Abrams[4] measured slips at the free end of the bar only; he considered 0.001 in. at the free end to be a significant value of slip suitable for comparisons. This value has since been used often. Some authors have, however, made comparisons at several values of slip, *e.g.*, 0.0005, 0.001 and 0.005 in. at the free end.

In 1935, Emperger[5] suggested that bond performance be based on the slip at the loaded end of a pull-out bar. He also stated that his tests had shown that the slip at the beginning of the anchorage in a beam seldom would exceed 0.01 in. at failure.

This criterion of performance was developed independently by Menzel[6]. Mr. Menzel, however, devoted most of his discussion in that paper to slip at the free end. Later, he and several others have based comparisons on the value of 0.01 in. slip at the loaded end. Clark[7] introduced an average steel stress at several values of slip as a basis of comparing different bars. He placed the main emphasis on slips up to 0.005 in. at the loaded end.

Little is known about the magnitude of slip which is most significant in structural members. Abrams[4] found slips in beam tests from 0.001 to 0.01 in. up to the load at which excessive deformations took place. Slips were measured at several points on the beams, not just at the free ends. Later investigators have found a similar order of slip values. Thus, it seems reasonable to believe that the values of 0.003- to 0.005-in.

slip at the loaded end of a pull-out bar are suitable for the comparisons necessary to evaluate the effect of air entrainment on bond.

In the tests described herein, after considering some slips at the free end, slips at the loaded end in the region of 0.001 to 0.005 in. were finally used as the main basis for comparing bond performance.

Compressive strength and modulus of rupture

Fig. 6 shows the effect of air content on the compressive strength and modulus of rupture of the concrete. The trends of these curves are in accordance with what might be expected from earlier tests by others, the compressive strength decreasing much faster than the modulus of rupture with increasing air entrainment above about 4 percent. At 7.5 percent air, the compressive strength was reduced 29 percent, and the modulus of rupture by 22 percent. In this connection, there is reason to believe that increased workability due to the same air entrainment has a greater beneficial effect on the flexural than on the compressive strength.

Vertically-cast bars

The bars cast in a vertical position were intended to yield information mainly concerning those effects of air entrainment on bond which accompany a reduction in compressive strength. Nevertheless, even though the bars were pulled in the opposite direction to the settlement during casting, some effect of changes in workability and in settlement under the lugs was present.

The slip at the free end of the ½-in. Hi-Bond bars cast in a vertical position was too small to be significant (less than 0.001 in.) when the pull-out tests were discontinued at the yield point. Also, the slip at the loaded end remained smaller than the value of 0.01 in. most fre-

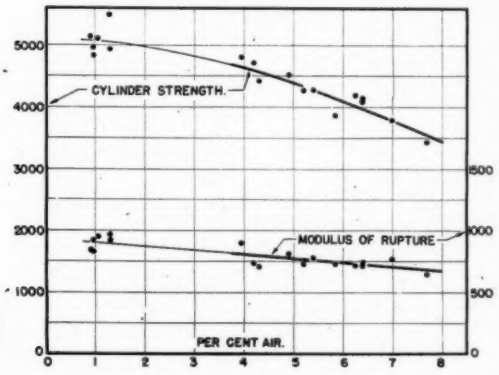

Fig. 6—Effect of entrained air on 28-day strength of concrete

quently used in the earlier tests. A slip of 0.003 in. at the loaded end was therefore chosen as a basis of comparison.

In earlier tests,[7] bars have been rated on the basis of their average performance over some definite range of slip. A similar concept was adopted in these tests by defining the rating of a ½-in. bar as the average bond at 0.001, 0.002 and 0.003 in. slip at the loaded end. Such ratings were derived from the bond-slip curves.

The bearing areas per unit surface area of the ½-in. and ⅞-in. Hi-Bond bars being nearly equal, the quality of the concrete the same, the slip at the free end of the bars small, and in view of the geometrical similarity of the test specimens, it was considered likely that the slip at the loaded end of the bars, for identical steel stresses, would be in proportion to the factor of similarity. Therefore, slips at the loaded end for the ⅞-in. Hi-Bond bars should be near 7/4 of those for the ½-in. Hi-Bond bars at the same applied steel stress.

Conversely, if ½-in. bars are studied as before on the basis of 0.003-in. slip, and the ⅞-in. bars on 0.00525-in. slip, then the corresponding average bond values should be nearly equal. Similarly, the ratings for the ½- and ⅞-in. bars should give nearly equal average bond stresses. The ratings for the ⅞-in. bars were computed as the average bond at 0.00175, 0.0035 and 0.00525-in. slip at the loaded end. The Laclede bars were studied in a similar manner.

The results obtained from the two sizes of Hi-Bond vertically-cast bars, Fig. 7, group themselves along curves as predicted by the above hypothesis. The bond strength decreased with increasing air content up to about 4.5 percent, but remained constant between 4.5 and about 7.0 percent, the maximum obtained in this group of tests. This leveling off of the bond strength is believed to indicate that the increased workability due to air entrainment improved the "grip" of the concrete on the bar and thus compensated for the reduction in bond strength which would be expected to accompany the reduction in compressive strength due to air entrainment.

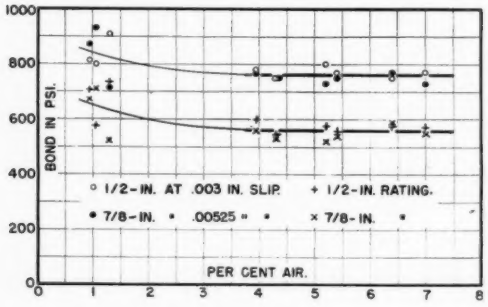

Fig. 7—Effect of entrained air on bond of vertically-cast Hi-Bond bars

There is some tendency toward higher bond strengths for the ½-in. bars, corresponding to the slightly higher relative bearing area, but the difference in actual lug height, and lug height to diameter ratio, seem to be of minor importance for these vertically-cast bars.

The values for the Laclede vertically-cast bars (Fig. 8) were uniformly reduced by about 10 percent at 4.5 percent of entrained air, and continued to decrease about 2 percent for each one percent of air. Thus, better workability due to increased air content seemed to have had less beneficial effect on the Laclede than on the Hi-Bond bars. As before, the results group themselves along definite curves; but in this case with the ⅞-in. bars in a slightly higher position. The scatter is somewhat greater than for the Hi-Bond bars.

The effect of air entrainment may best be studied in Fig. 9 in which the curves from Fig. 6, 7 and 8 are redrawn with ordinates equal to the percentages of the average bond strengths for six batches of non-air-entraining concrete. All air-entraining mixtures contained between 4 and 8 percent of air. The curves between 1 and 4 percent are shown as thin lines indicating a possible trend not verified by the tests.

The bond for a specified slip is reduced by 10 percent for both types of bars at 4.5 percent of air. The compressive strength is also reduced

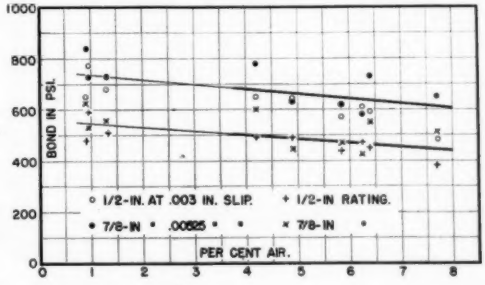

Fig. 8—Effect of entrained air on bond of vertically-cast Laclede bars

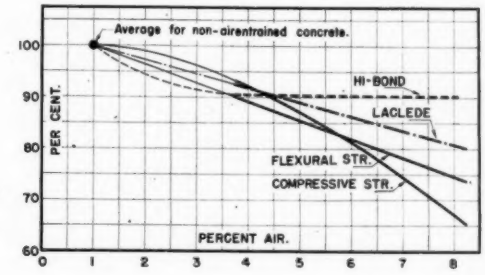

Fig. 9—Comparison for vertically-cast bars

by a similar amount. For air contents above 4 percent, the bond values for the Hi-Bond bars remain constant, while those for the Laclede bars continue to fall off.

Other deformed bars meeting A.S.T.M. A 305-47T must be expected to have lug characteristics not much different from the Laclede and the Hi-Bond bars. Therefore, it can be concluded that the percentage reduction in bond of such vertically-cast bars, due to an air entrainment of 4 to 5 percent, can be expected to be about the same as the corresponding reduction in compressive strength. For entrainment of 5 to 8 percent of air, however, the bond will be reduced less than the compressive strength and the modulus of rupture.

Horizontally-cast bars

Within the limits of these tests, the following main variables are believed to affect the bond performance of a reinforcing bar cast in a horizontal position: lug height, lug spacing, lug profile, bearing area of deformations, size of bar, depth of concrete under the bar, and percent of entrained air.

Previous investigators have noticed and emphasized the effects and importance of settlement of fresh concrete under horizontally cast bars. Abrams[4] discussed the phenomenon that a horizontally cast pull-out bar, which is held rigidly in position during casting, will show inferior bond properties. Mr. Abrams emphasized this fact without considering the effects of various concrete depths under the bar; nor did he state firmly how important this effect of settlement is for practical purposes.

In 1928, F. E. Richart reported cavities under the negative reinforcement in some test beams. These cavities reduced the bond between concrete and steel, and thus were believed to have been the cause of a serious slipping observed at low loads. The same year Edwards and Greenleaf[8] published some tests in which pull-out prisms 8 x 8 x 12 in. were cast with the bar held in a horizontal position. In discussing that paper, Professor Richart emphasized the effects of settlement of the fresh concrete under the bars and stated that a good type of test piece for bond tests would be one with horizontal bars near the top of a beam section.

The fundamental importance of the effect of depth of concrete under a horizontally-cast bar, however, does not seem to have been discussed before Menzel's papers.[6,9] Since then several investigators have taken this variable into consideration. At the present time the importance of settlement conditions are fully realized, and a test of bond properties would not be complete without consideration of horizontally-cast bars.

For the horizontally-cast bars, the bond stresses at 0.003- and 0.00525-in. slip at the loaded end for the ½- and ⅞-in. bars, respectively, were used in these tests as a quantity to denote performance. The results

showed that the slips at the free end for such bond stresses were no longer negligible, as was the case for the vertical bars. This was especially true for the high air contents. Furthermore, the effect of lug height and depth of concrete under the bars must be expected to be considerable. Thus the concept of geometrical similarity would not be expected to apply.

Because of the increased scatter obtained for the horizontal bars, it was felt that little could be gained by a study of ratings, as was done for the vertical bars, and such studies were not made.

Fig. 10 shows the bond stresses of horizontally-cast ½-in. Hi-Bond bars corresponding to slips at the loaded end of 0.003 in. The stresses for vertically-cast bars are also indicated. The former curves show a considerable increase in bond for 4 percent entrained air. The increase is explained by a beneficial effect of air entrainment on the settlement of the concrete. Beyond 4 percent, the bond decreased about 10 percent for each one percent of entrained air.

The curves in Fig. 11 show the bond stresses at 0.00525-in. slip for the ⅞-in. Hi-Bond horizontally-cast bars. There is practically no change in the bond values as the air increased to about 4 percent, indicating

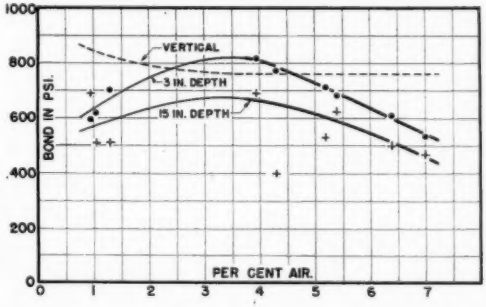

Fig. 10—Effect of entrained air on bond of horizontally-cast ½-in. Hi-Bond bars at 0.003-in. slip for 3- and 15-in. concrete depth under bars

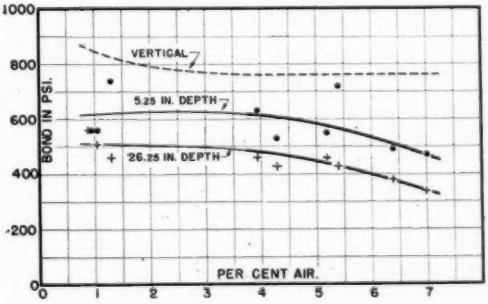

Fig. 11—Effect of entrained air on bond of horizontally-cast ⅞-in. Hi-Bond bars at 0.00525-in. slip

also a beneficial effect of air entrainment on settlement. Some of this effect must have been present, since the compressive strength and the bond for the vertically-cast bars decreased about 10 percent. Beyond 4 percent, the bond decreased from 8 to 10 percent for each one percent increase of air.

The average bond at a specific slip for non-air-entraining concrete was about 840 psi for both sizes of Hi-Bond bars cast in the vertical position. Similar stresses for the horizontal ½-in. bar with 3-in. depth and ⅞-in. bar with 5.25-in. depth decreased about 25 percent to 630-620 psi. Due to the greater depth of concrete under the bar, the ⅞-in. bar should have been expected to suffer the greater reduction. Apparently the higher lugs of the ⅞-in. bar counteracted the reduction. For a depth of 15 in., the bond for the ½-in. bar was reduced about 32 percent to 570 psi, while that for the ⅞-in. bar with 26.25 in. of concrete under it was reduced about 40 percent to 510 psi. This again indicates an effect of lug height. The ½-in. bars, due to a smaller lug height, seem to be more sensitive to the effect of settlement under the bar.

Considering the absolute values of bond at 4 percent air, the ½-in. bar with 3 in. concrete depth rates highest with about 800 psi. The ⅞-in. bar with 26.25-in. depth rates lowest with about 480 psi. This is an effect of settlement. An approximately equal value, about 670 to 620 psi, was found for the ½-in. bars with 15-in. depth and the ⅞-in. bars with 5.25-in. depth. The ½-in. bar with 15-in. depth should be expected to be inferior, both because the lugs are smaller and because the depth is larger. However, since the slips at the free end of the ⅞-in. bars were larger than those for the ½-in. bars, the results obtained seem to be reasonable.

Fig. 12 shows the bond stresses for the ½-in. horizontally-cast Laclede bars at 0.003-in. slip. These curves show little change in the bond values as the air content increased to about 4 percent, but beyond that the bond decreased from 5 to 10 percent for each one percent of air. The

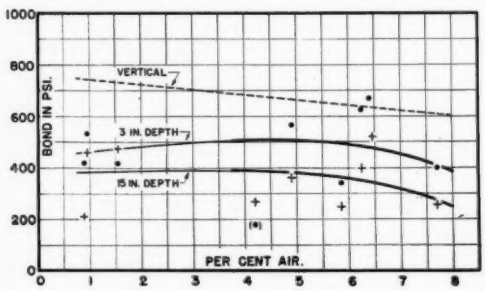

Fig. 12—Effect of entrained air on bond of horizontally-cast ½-in. Laclede bars at 0.003-in. slip

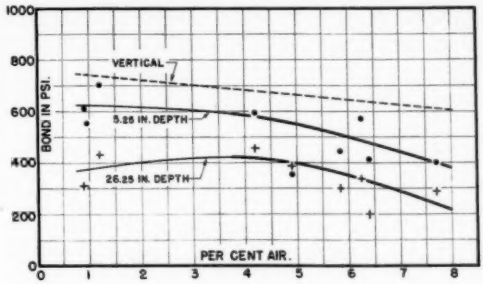

Fig. 13—Effect of entrained air on bond of horizontally-cast ⅞-in. Laclede bars at 0.00525-in. slip

reductions for these bars are believed to depend mainly on the same three variables as affected the Hi-Bond bars; namely lug height, size of bar and depth of concrete under the bar.

Fig. 13 shows the bond stresses at 0.00525-in. slip for the ⅞-in. horizontally-cast Laclede bars for various amounts of entrained air. Little change in the bond occurred for air contents up to 4 percent. Beyond 4 percent, the average decrease was about 12 percent for each one percent increase of air.

Since the reduction in bond for air contents greater than 4 percent, as previously shown, was in general greater than the reduction in compressive and flexural strength, it is believed that considerable effect of rising air under the horizontally-cast bars was present when the air content exceeded 4 percent. By splitting some specimens after test, it was also found that the mortar under the bar appeared to have a large content of air even though no large cavities were present.

At 4 percent of air, the bond for all upper horizontally-cast bars ranged from 17 to 28 percent less than that for the corresponding bars cast in the lower position.

SUMMARY

The influence of air entrainment on the bond properties of deformed bars, as indicated by these tests, depends on several characteristics of the reinforcing bars. At slips of about 0.003 to 0.005 in. at the loaded end of a pull-out bar, the percentage reduction in bond caused by about 4 percent of entrained air, as compared to non-air-entraining concrete, was equal to or less than the reduction in cylinder strength, for all four bars and all positions of casting used in these tests. Since air entrainment has an important bearing on the settlement of the concrete which occurs shortly after pouring, the vertically- and horizontally-cast bars are discussed separately.

Vertically-cast bars

At a given slip, bond stresses of vertically-cast bars were found to increase with the bearing area per unit of surface area. Other variables, such as height and spacing of lugs, were found to be of minor importance.

Using geometrically similar specimens, bond stresses of vertically-cast bars, at geometrically similar slips at the loaded end, were found to be independent of the size of bar. This applies to slips of 0.003 to 0.00525 in. at the loaded end, if the slips at the free end are negligible.

Compared to non-air-entraining concrete, the bond at a specific slip was reduced about 10 percent when 4.5 percent of air was entrained. This reduction was about the same as the corresponding reduction in compressive strength and modulus of rupture of the concrete. When over 4.5 percent of the air was entrained, the bond of the Hi-Bond bars remained constant, while that of the Laclede bars continued to decrease about 2 percent for each one percent of entrained air; this was much less, however, than the rate of reduction of the compressive strength of concrete.

Horizontally-cast bars

At a given slip, bond stresses of horizontally-cast bars were found to increase with the bearing area.

The height of lugs was an important variable. Bond stresses increased with the height of lugs.

Bond stresses decreased with increased depth of concrete under the bars.

Air entrainment of 4 percent caused little change in bond as compared to non-air-entraining concrete; for the ½-in. Hi-Bond bars the bond even increased. For air entrainment over 4 percent, a reduction in bond took place. This reduction in terms of the values at 4 percent air, ranged from 5 to 15 percent for each one percent of air over 4, while the compressive strength was reduced about 6.5 percent for each one percent of air over 4.

The average reduction for all horizontal bars, as compared to non-air-entraining concrete, was 10 percent at 6 percent air and 20 percent at 7 percent air. The corresponding reductions in cylinder strength were 19 and 25 percent. Thus the percentage reduction in bond for a specified air content of 3 to 6 percent was generally less than the reduction in compressive strength. Nevertheless, it is recommended that the air content not exceed 5 percent if large depths of concrete are placed under the reinforcing bars.

ACKNOWLEDGMENTS

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Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Design of prestressed concrete

Modern Developments in Reinforced Concrete, No. 25, Portland Cement Assn., Chicago, 1950, 15 pp.

Avoiding consideration of the theoretic aspects of prestressing, this booklet illustrates the actual calculations required in the design of a structure utilizing its principles. Taking a bridge deck as an example, a step by step design procedure is worked out. A bibliography lists 65 references.

The analytic shaping of arches and bridges (Die analytische Formung der Bogen und Bogenbrucken)

K. SCHADEN, *Oesterreichische Bauzeitung* (Vienna), V. 4, No. 10, pp. 165-168

Reviewed by RUDOLPH FISCHL

In this paper, a differential equation of the first order is developed from the relations between the load and the pressure line. The solution of this equation is found by iteration. The shape of the arch is then developed after the exact pressure line. Examples are given for three-hinged, two-hinged and fixed arches.

Mixing of Korean black shale with portland cement

TOSHIYOSHI YAMAUCHI, *Journal of the Japanese Ceramic Assn.*, V. 51, No. 603, 1943, pp. 158-162

CERAMIC ABSTRACTS Feb. 1950

Korean black shale containing graphite, found in Kato Gun (Keisho Nando) was added to portland cement in amounts of 10, 20 and 30 percent, and the products were tested for water absorption. The sample containing 10 percent shale showed lower water absorption and higher strength than the sample without shale.

New materials and nontraditional methods of construction (Materiaux nouveaux et des procedes non traditionnels de construction)

Cahier du Batiment (Paris), No. 62, Oct. 1949

Building materials and construction methods described include bearing walls faced with cast stone backed up by lightweight concrete blocks, trussed forms for monolithic concrete and several prefabricated reinforced concrete ceiling systems. These materials and methods supplement those reported in *Cahier du Batiment*, No. 52, July 1949 (See p. 230, ACI JOURNAL, Nov. 1949).

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Design of variable sections in reinforced concrete (Beitrag zur Bemessung beliebig geformter Stahlbetonquerschnitte)

K. FISCHER, *Betonstein Zeitung* (Wiesbaden), 1949, No. 11, pp. 202-204 Reviewed by RUDOLPH FISCHL

Odd shaped cross sections frequently occur in the design of precast concrete members. Instead of using a rough approximation, it is more economical to design for the real section. The graphical method developed by Otto Mohr allows finding the moment of inertia for compound sections by elimination of the tension zone of the concrete. Two examples, one with tension reinforcement, the other with tension and compression reinforcement, are given.

Production and properties of concrete with cement and wood shavings (Ueber die Herstellung und uber die Eigenschaften des Betons aus Zement under Holspanen)

O. GRAF, Stuttgart, Bauverlag, Wiesbaden, 1949, 39 pp. Reviewed by RUDOLPH FISCHL

The tests reported in this booklet were made to find the fundamental conditions for production of concrete wall boards made of cement and wood shavings.

The tests cover the influence of the quality of cement on the flexural and compression stresses and specific weight and the quality, size, shape and pre-treatment of the wood shavings. The curing, shrinkage and durability of the boards were carefully examined.

The results of the tests are condensed in the last chapter as "Recommendations for the production of concrete with wood shavings."

Concrete masonry units, their properties and use (Betonghalblock och massiva betongblock)

Swedish Cement Assn. (Malmö), 1949, 63 pp. 1.50 Sw. Kr. Reviewed by EIVIND HOGNESTAD

The present Swedish manufacture of concrete masonry units, the majority of which are made with normal (not lightweight) aggregate, amounts to about 25 million pieces a year. This publication gives information regarding the properties and use of such masonry units; the problems of manufacture, however, are not discussed. The first chapter is devoted to such physical properties of concrete masonry units as strength, absorption, fire resistance and insulation. Tentative standards for the sizes and shapes of hollow units are also presented.

A second chapter presents many structural details of walls and foundations. The last chapter gives instructions regarding proper masonry work. A number of graphs are presented in an appendix giving the necessary thickness of various insulation materials as a function of the specific thermal conductivity (k).

Is new design practice needed for concrete floor slabs in dwelling houses? (Erfordras en ny konstruktionspraxis foer bostadsbjaelklag?)

V. HASSELBLAD and T. E. SUNDSTROM, *Betong* (Stockholm), V. 34, No. 4, 1949, pp. 223-238 Reviewed by EIVIND HOGNESTAD

In many dwelling houses in Sweden, the concrete floor slabs have developed large plastic deformations which have caused damage to partitions and necessitated expensive repairs. The authors argue that the cause of this trouble probably is the use of too-thin slabs. The present design standards, which limit the thickness of a two-way slab to 1/40 of the span, do not seem to be sufficiently effective in limiting the deformations of large concrete slabs. This is especially so since "the span" has been interpreted as the distance between points of contraflexure for continuous slabs. It is expected that a new method of design, known as the fracture line method, will be used to an increasing extent, and it is therefore the more urgent to develop a design practice which limits the plastic deformations. Basing their arguments on practical experience, the authors feel that the standard specifications relating to the thickness of two-way slabs should be rendered more severe. A minimum thickness of 1/35 times the smallest side length of two-way slabs is suggested.

Calcium hexaluminate in the system $CaO—Al_2O_3—SiO_2$

N. E. Filonenko and I. V. Lavrov, *Doklady Akad. Nauk S.S.S.R.*, V. 66, No. 4, 1949, pp. 673-76

Ceramic Abstracts Jan. 1950

Microscopic study of rapidly chilled samples of the high-alumina part of the system $CaO—Al_2O_3—SiO_2$ reveals the existence of corundum, $CaO \cdot 6Al_2O_3$ and $CaO \cdot 2Al_2O_3$. The $CaO \cdot 6Al_2O_3$ crystallizes the hexagonal system and separates from the melt in hexagonal plates which comprise a combination of basal pinacoid forms with a bipyramid and rarely with a prism; $n_\omega = 1.759$ and $n_\epsilon = 1.752$. The point at which the corundum, $CaO \cdot 6Al_2O_3$ and anorthite exist in equilibrium with the liquid and vapor has the composition Al_2O_3 41.0 ± 0.5 percent, SiO_2 36 ± 0.5 percent, and CaO 23.0 ± 0.5 percent, the melting point is 1495 ± 5°. The field of $CaO \cdot 6Al_2O_3$ borders on the fields of stability of corundum, anorthite, gehlenite, and $CaO \cdot 2Al_2O_3$. $CaO \cdot 6Al_2O_3$ melts with decomposition into corundum and liquid. The reaction $CaO \cdot 6Al_2O_3 \rightleftarrows$ corundum + melt takes place at a definite concentration within the interval 1500 to 1850 C.

A dynamic method for the determination of the speed of sound in solids (En dynamisk metode til bestemmelse af lydhastighet i faste stoffer)

J. Foss Brink and S. Christensen-Dalsgaard, *Ingenioeren* (Copenhagen), V. 59, No. 2, Jan. 1950, pp. 50-54

Reviewed by Eivind Hognestad

This paper presents a method for the determination of the velocity of longitudinal sound waves in elastic solids. Such methods are used to examine dynamic properties of building materials, especially concrete. The method is suitable for laboratory tests as well as tests of structures in the field.

The apparatus used consists of a condenser chronograph (Joergensen-Weibel system) in connection with two piezo-electric gramophone pick-ups. Some directions for the practical performance of the measurements are given, and limitations and sources of error are discussed.

An aim of the studies presented is the determination of the strength of concrete by nondestructive means.

Reinforced concrete wall and column footings (Vegg- og soylefundamenter i armert betong)

Eivind Hognestad, *Teknisk Ukeblad* (Oslo), V. 96, No. 49, Dec. 8, 1949, pp. 921-929

Author's Summary

As the Norwegian specifications for reinforced concrete footings differ somewhat from the recommendations of the Joint Committee and the American Concrete Institute, the author outlines the development of these American standards from Talbot's basic work to the present status. The major part of the paper, however, is devoted to an analysis of the present Norwegian specifications in the light of the 1944-48 tests at the University of Illinois, and it is concluded that a satisfactory agreement exists. The main difference between these Norwegian specifications and the present ACI Code is the use of full static moments and shears in the former, 85 percent in the latter. Furthermore, the Norwegians assume the section involved in estimating diagonal tension at a distance $\frac{2}{3}\,d$ from the face of columns, pedestals or walls, while the American codes use d or $\frac{1}{2}\,d$ depending on whether the footings rest on soil or piles.

Differential thermal analysis

Marjorie J. Vold, *Analytical Chemistry*, V. 21, No. 6, 1949, pp. 683-88

Ceramic Abstracts Feb. 1950

Differential thermal analysis is essentially a refinement of the classical procedure of studying phase transformations by means of time-temperature records during uniform

heating or cooling of the system. The construction and operation of a fully automatic self-recording differential calorimeter is described and the factors influencing the design finally adopted are discussed. The experimental procedure consists in heating or cooling the sample side by side with an inert reference material in the same furnace and measuring both the sample temperature and the temperature difference between sample and reference material as a function of time. When a phase temperature change occurs involving absorption or evolution of heat, the temperature difference between reference and sample begins to increase; after the transformation is complete, the temperature declines again. Thus each transformation produces a peak in the curve of temperature difference against time. Equations are derived which make possible the calculation of heats of transformation from differential heating curves, independent of external calibrations.

The importance of quickly covering green concrete in cold weather (Om betydningen af hurtig tildaekning af beton stoebt om vinteren)

ERIK RASTRUP, *Ingenioeren* (Copenhagen) V. 59, No. 2, Jan. 1950, pp. 60-62 AUTHOR'S SUMMARY

In the "Tentative Recommendations for Winter Concreting Methods" proposed by the Concrete and Reinforced Concrete Section of the Danish Institution of Civil Engineers, and issued by the Danish National Institute of Building Research as Direction No. 2, 1948, a number of diagrams show the temperature to which concrete under various conditions should be heated to avoid being damaged by frost during the hardening. In these diagrams it is assumed the concrete is covered by various insulating materials immediately upon placing and compaction.

However, in practice it is almost impossible in some cases to place the cover without some delay. To get an impression of what this delay entails and to enable the contractor to compensate correctly for unavoidable delays, the diagram in Fig. 3 was established. Starting with any thickness of the concrete slab it gives, for delays from five minutes to four hours and for all practical values of the difference in temperature between concrete and air, the initial extra heat necessary to counteract the delay.

The closing paragraphs of the paper supply the theoretical formulas on which the diagram is based.

Modern concrete floors (Modern betonggolvteknik)

V. BAHRNER, Swedish Cement Assn. (Malmö), 1949, 151 pp. 4.00 Sw. Kr.

Reviewed by EIVIND HOGNESTAD

The present publication is devoted to the many problems involved in making the wearing surfaces of concrete floors. Such surfaces are classified according to various degrees of mechanical wear without or combined with some destructive action of chemicals. A chapter of about 30 pages gives a general description of the proper manufacture of concretes and mortars to be used for wearing surfaces. Numerous methods for compacting and finishing the surface materials are also described in detail.

The next chapter presents a variety of methods for the construction of floors without a special wearing surface and cast directly on the subsoil. Later chapters deal with such problems as the construction of floors with a special wearing surface, floors cast on various types of insulation materials, basement floors subject to ground water pressures, floors with parquet or linoleum finish, etc.

Special wearing surfaces are then described, followed by a presentation of methods of preventing dust formation, the effect of various chemicals on concrete, methods of grinding and polishing concrete, and proper procedures for patching and resurfacing old floors.

Finally, specifications are given for a variety of wearing surfaces and their base.

The publication contains a large amount of knowledge and practical experience, well classified for easy reference by the user of concrete.

Stabilization of foundation by cement injection (Stabilisatie van fundeering door cement-injecties)

J. F. Bish, *Cement* (Amsterdam), V. 1, No. 7-8, 1949, pp.125-126 Reviewed by J. W. T. Van Erp

For a large lathe with a mobile part weighing 96 tons it became imperative to stabilize the foundation. Impermissible variations in settlement occurred when the mobile part was moved from one end to the other, thereby increasing the labor of continual resetting and readjustment by almost 100 percent. As the machine, the largest of its kind in Europe, was in continuous operation, a method of reinforcing its foundation which would least interfere with its operation was desirable. Of the various schemes considered the best seemed to be to drive piles alongside the existing foundation and extend the foundation over it. Available space and headroom, however, proved to be insufficient. The adopted method was to reinforce and stabilize the subsoil by cement injection.

The subsoil consisted of fine sand on which a load of 2-2.5 tons per sq ft could normally be allowed. Its porosity was determined and thereby the amount of cement sludge to be injected could be estimated. Care was taken to inject no air as this would lead to failure of the method. Cement sludge was injected from a container in which it was kept in continuous motion, through perforated pipes into the soil. Injection in every pipe was started with sludge of a fluid consistency and ended with a stiffer mix, this giving an ultimate bearing action similar to that of a pile foundation.

The complete job proved to be entirely successful and was considerably less expensive than other possible methods.

Concrete chimneys (Gewapend beton schoorsteenen)

S. A. Timmers, *Cement* (Amsterdam), V. 1, No 7-8, 1949, pp. 127-34 Reviewed by J. W. T. Van Erp

A number of concrete chimneys were constructed for the Shell Oil Co. refinery at Rotterdam. Although sheet steel stacks were usual, concrete was selected because of the scarcity of steel. Steel stacks have a life expectancy similar to the rest of the plant equipment, which sometimes is an advantage in view of the rapid turnover and replacement of plant in this industry, often entailing also a change in the stack requirements.

The chimneys were built on pile foundations, the piles being 75 ft long and having a bearing capacity of 41 tons. The stacks were designed for a wind pressure of $\frac{2}{3}$ (24 + 0.04 h) psf on the projected area, in accordance with official wind load standards. Temperature of the incoming gases at the base is 300 C, and may occasionally rise to 470 C. The inside diameter tapers down by 4 ft over its entire height. The concrete wall is 8 in. thick at the base, 4 in. at the top. Reinforcement is double up to 60 ft and single above. Adjustable forms were of sheet steel. A material hoist was located inside and stood detached from the stack. The firebrick lining over the entire height was built in 30 ft sections supported on prefabricated (in 60 sections) ringbeams resting on concrete supports jutting out from the mantle.

All concrete was completely protected by firebrick. The inside of the concrete mantle was first covered by 2 coats of flintcote. Then the firebrick lining was built and the $2\frac{3}{4}$-in. cavity was then backfilled with granular vermiculite lightly tamped in place. To prevent excessive settlemont in the loose vermiculite fill the firebrick wall had projecting layers into the cavity every 6 ft 6 in. Instead of vermiculite, cinders have also been successfully used as a backfill. The chimneys are capped by a lead-covered cast iron segmental cap.

Prestressed concrete

P. W. ABELES, Crosby Lockwood and Son, Ltd., London. American Edition, Frederick Ungar Publishing Co., New York, 1949, 112 pp. $3.75

Reviewed by GLENN MURPHY

This attractive and well organized book is based on a series of articles by the author published in "Civil Engineering and Public Works Review" during 1948. It does not, however, represent a mere reprinting of those articles, as the author has extended the scope and reorganized the material to form an excellent presentation. The combination of Dr. Abeles' enthusiasm for the subject and his extensive experience in the design and testing of prestressed concrete has resulted in a practical and readable treatise.

The text opens with a clear presentation of the basic idea of prestressing in general followed by a discussion of its application to reinforced concrete in particular. Careful distinction is made between pre-tensioning and post-tensioning as two methods of attaining prestressed beams. Loss of prestressing due to transfer technique, creep in the concrete and other effects are discussed. An impartial description of the various systems and methods of prestressing completes the introduction.

The section on analysis of prestressed members brings together the results of a number of writers and leads directly to the chapter on design. Simple bending, bending with respect to two axes, compressive reinforcement, and combined sections are among the topics treated. Examples of designs, together with test results, follow.

A number of pages, with excellent photographs, are devoted to the practical details of pre-tensioning, followed by a similar section on post-tensioning. A short chapter on the economy of prestressing is included, in which data are given for conditions in England. American readers may make some interesting comparisons from the prices quoted.

The last chapter in the book is an 8-page summary and a set of conclusions calling attention to the structural advantages of prestressing. An extensive list of references is included.

New contribution to the study of shrinkage of cements (Nouvelle contribution a l'etude du retrait des ciments)

R. L'HERMITE, J. CHEFDEVILLE and J. J. GRIEU, *Memoires sur la Mecanique-Physique Du Beton III, Liant Hydrauliques*, No. 5, Annales de L'Institut Technique du Batiment et des Travaux Publics, No. 106, Dec. 1949, 28 pp.

This investigation is included in a series of reports on the mechanics of concrete published in the Annales du Batiment et des Travaux Publics and is really a sequel to a paper entitled "The Shrinkage of Cement, Mortar and Concrete." In this earlier paper, dated June 1947, the author first reviewed all the experimental work carried out in various countries on the causes and results of shrinkage and its measurement, and then outlined a theory of shrinkage based on thermodynamics and connected with the fact of evaporation. He then dealt with the effect of the composition of various kinds of concrete on shrinkage and the occurrence of internal stresses as a result of these deformations; these points again, being treated in a purely theoretical manner. It was chiefly to give support to these theories and to give values to the numerical constants suggested, that the second paper was written. It was the result of a large number of long-time experiments, but only bears on a few special points among those developed in the first paper. The effect of the humidity maintained around the test pieces is dealt with very fully and also the effect of adding an inert mineral powder to the cement.

It is demonstrated that free water in cement evaporates in accordance with Fourier's law, and this can be represented approximately by a simple formula which is a function of the size of the test piece and the time of exposure to the air, the degree of humidity and the quantity of evaporable water and also of a coefficient of evaporation which is

itself a function of the age of first exposure to the air and for which numerical values are suggested varying with the composition of the paste used.

As concerns shrinkage, its formula comprises a coefficient of deformation which accounts both for a deformation reversible by immersion in water, and for a permanent deformation which forms gradually as if caused by creep.

The results of an investigation into alternate shrinkage of specimens kept first in water then in the air give some idea of the reasons why deformation occurs. This work is continuing and it is to be assumed that further details on the subject will follow.

Plain concrete

EDWARD E. BAUER, McGraw Hill Book Co., New York, 1949, Third Edition, 441 pp. $5.00 Reviewed by MARY KRUMBOLTZ HURD

With new illustrations, data and definitions, Professor Bauer has brought up to date this combination text book and laboratory manual. In a work designed to give the engineering undergraduate a professional background in the field of concrete, several features are conspicuously valuable for the student. A set of review questions follows each chapter, and generous footnote references to recent periodical literature enable the student to expand his study with the latest available information. Style of presentation is easy; highly readable for either layman or student. Each new term is carefully but nonpedantically defined as it appears. Illustrations are modern and in general effectively chosen. The student is introduced to the standards-making bodies in the concrete field, and their material is freely used.

Part I, comprising 65 percent of the book, relates the history of cement usage, describes the various manufacturing processes for standard portland cement and discusses physical tests for cement. The entire A.S.T.M. specification C 150-47 for cement is quoted. Special cements are described and evaluated; complete A.S.T.M. C 175-48T for air-entraining cements is given. Further chapters devoted to admixtures and aggregates quote A.S.T.M. C 33-46 for aggregate. Professor Bauer defines and classifies aggregates in terms of composition, grading, physical properties and testing.

Theories of proportioning are traced from the empirical stage through the Fuller-Thompson studies shortly after the turn of the century. Then Abrams' studies and the resulting W/C ratio law are presented; then Edwards' work in 1918 toward proportioning on a basis of surface area of graded and uniform aggregate; Young's application of Edwards' ideas using Abrams' W/C ratio, but computing the amount of aggregate on the basis of surface area rather than fineness modulus; the Talbot-Richart studies of 1922; and the trial method whereby the mix may be designed as a job progresses. Detailed examples with sample calculations are given for the Fuller-Thompson, Abrams and Talbot-Richart methods.

In a section on applied proportioning, Bauer discusses general problems and provides sample calculations for nearly all factors in design and control of mixes including air content and field batch weights. Both Illinois and Michigan modifications of the Talbot-Richart method are developed.

Factors affecting concrete strength are enumerated and explained. Handling and storage of materials, measuring and mixing are treated. In a chapter on placing, finishing and curing such unusual uses of ornamental concrete as John Earley's sculpture are mentioned. Durability, workability, waterproofness and problems of field control are covered at some length. Complete A.S.T.M. standards for ready-mixed concrete and for sampling of cement, aggregate and fresh concrete are quoted. The final chapter of Part I outlines procedures and describes equipment for ordinary testing problems.

Part II, a veritable laboratory manual, quotes complete A.S.T.M. standards for materials and mix tests. Where there is no A.S.T.M. standard method, the author gives complete directions of his own; *e.g.*, tests for unit weight, surface moisture, air

content by Indiana, Ohio and pressure methods. Problems and questions for the student supplement many of these tests, and Professor Bauer has also provided detailed direction for six original concrete projects emphasizing proportioning.

Prestressed concrete hangars (Vliegtuigloodsen in spanbeton)

H. C. Duyster, *Cement* (Amsterdam), V. 1, No. 7-8, 1949, pp. 117-124 and *De Ingenieur* ('s-Gravenhage), V. 61, No. 18, 1949, pp. 37-48 Reviewed by J. W. T. Van Erp

The articles describe hangars recently constructed on Melsbroek airdrome in Brussels. The large hangar spaces surrounded by series of smaller workshops primarily serve the maintenance of commercial airplanes and are part of the large extension program of that airport. The executed design was selected after comparative studies of different designs in concrete and other materials. Wood, although often used for hangar construction after World War I, was ruled out because of its combustibility, while a steel structure was rejected because of its vulnerability to fire and the difficulty of adequately fireproofing light structures of large span. While reinforced thin shell or arch construction has been used for this type of structure many times, flat-roof girder construction with a 167-ft span is the biggest ever executed in Europe. The results are considered economically favorable so that it is to be expected that greater spans in prestressed concrete will be built.

The roof girders run parallel to the front (opening) side of the hangars and are of box (trapezoid) section, with the smallest side at the bottom, this being the most favorable cross section for prestressed roof girders where the live- to dead-load ratio is relatively small. The prestress is such that under the maximum combined dead and live load the concrete stress in the bottom fibers will be approximately zero, while on the other hand the maximum compressive stress in that fiber under dead load and the prestress alone will not exceed the maximum allowable stress. The height of the girders varies from 1/20 to 1/18 of span. The ¼-in. steel wire reinforcement is contained within the hollow space of the girder and lies on its bottom. The reinforcement is prestressed to 127,000 psi. The prestress is checked in the first place by the pressure gages on the jacks and secondly by the considerable degree of elongation of the wire (about 10 in.). After the stretching operation, the wires are anchored at their ends by wedges (Belgian system) and covered by concrete over their full length, not as a bond, but primarily as rust and fire protection. In this respect it should be mentioned that this type of prestressed girder of box section has certain advantages as to fire-resistivity without losing its strength, an advantage particularly important in hangar structures, with their inherent fire hazard. In case of fire, the lower parts of the girders are exposed and it is exactly this part of the prestressed girder in which the concrete stresses are nearly zero (at the center of the span). A further advantage of the box section is that it has great rigidity and resistance against buckling. During the stretching of the reinforcement the girder is subjected to considerable eccentric longitudinal loads because wires must be stretched in small groups, simultaneously.

Another difficulty in concrete hangar construction is that the relatively great height of the roof requires much formwork high above grade. In this job the difficulty was solved by making the girders on ground level and later raising them into position by jacks. The hydraulic jacks were fixed at consecutively higher stages against the already constructed concrete columns. The weight of each girder is 300 tons.

The roof area is thus formed by the wide top section of the box girders supplemented by hollow precast slabs that span from girder to girder directly. These slabs were hollow to increase their heat insulating value, which was important because the hangars were designed and equipped for heating in the winter. For this reason also an elaborate grid of expansion joints was used. Expansion joints in the hangar area are located at the ends of the girders and in the surrounding workshops at 84 ft centers.

Recommended allowable unit stresses in concrete reinforced with new-style deformed bars

Report of ACI Committee 208 on Bond Stress*

HERBERT J. GILKEY
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SYNOPSIS

A brief review of committee action on bond stresses in reinforced concrete with a tabulation of recommended values for various types of reinforcing bars.

At its meeting in New York in February, 1949, Committee 208, Bond Stress, formulated and approved the bond stresses, given in the accompanying tabulation, as its version of what is reasonable for the types of bars and conditions set forth.

This approval was based on the recommendations of Committee 208, Subcommittee 2, (Frank E. Richart, Chairman) in the light of its careful continuous study of new test data on bond, as fast as such data became available to the subcommittee for study. The recommendations were conditioned upon certain pending revisions in A.S.T.M. Designation A 305 for deformed bars. These materialized in June.

On behalf of Committee 208, I respectfully recommend:

1. That this report be formally transmitted to Committee 318, Building Code, for its (continued) consideration in connection with possible building code revisions.
2. That this report be approved by TAC for formal publication in the JOURNAL as information, but with the following qualifying comment:
3. These Committee 208 suggestions are subject to possible approval, alteration or rejection at any of the succeeding stages

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through which they must pass before becoming a part of any Institute Standard or Code. They are what they purport to be and nothing more: recommendations from one Institute committee (208) offered for the optional guidance of another Institute committee (318), as Committee 318 studies possible alterations in the code for which it initiates recommendations for change.

The essence of the table which follows was presented informally to the membership of the Institute as a *News Letter* release* at the time Committee 208 deemed its assignment to be complete and in the hands of Committee 318 for such consideration as Committee 318 sees fit to accord.

*ACI JOURNAL, Apr. 1949, *News Letter*, p. 14.

ACI COMMITTEE 208, BOND STRESS— RECOMMENDATIONS FOR ALLOWABLE UNIT STRESSES IN CONCRETE

Description	u for any strength of concrete	Allowable unit stresses				
		Ceiling value, psi, not to be exceeded	f'_c			
			2000	2500	3000	3750
Bond: u						
Deformed bars†—Straight or hooked ends, exclusive of top bars as defined below.						
a) In beams, slabs; one way footings	$0.10f'_c$	350	200	250	300	350
b) In two-way footings	$0.08f'_c$	280	160	200	240	280
Plain bars						
a) In beams, slabs; one way footings (must have hooked ends)	$0.045f'_c$	158	90	113	135	158
b) In two-way footings (must have hooked ends)	$0.036f'_c$	126	72	90	108	126
Top bars—Bars, near the top of beams and girders having more than 12 in. of concrete under the bars						
a) Deformed bars	$0.07f'_c$	245	140	175	210	245
b) Plain bars (must have hooked ends)	$0.03f'_c$	105	60	75	90	105

†Deformed bars must comply with A.S.T.M. Specification A 305 as any bars not meeting this specification shall be classed as plain bars.